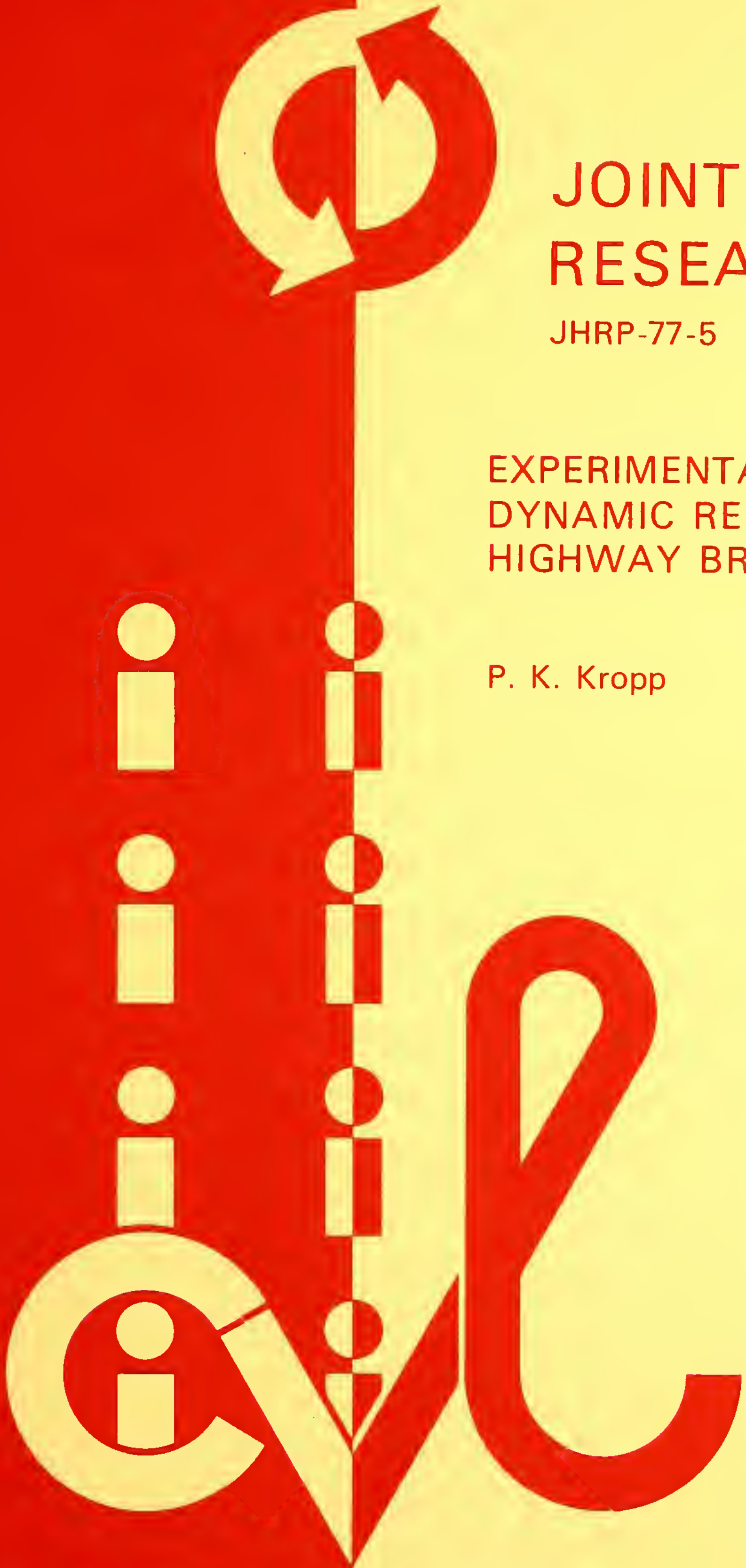
SCHOOL OF CIVIL ENGINEERING



JOINT HIGHWAY RESEARCH PROJECT

JHRP-77-5

EXPERIMENTAL STUDY OF THE DYNAMIC RESPONSE OF HIGHWAY BRIDGES

P. K. Kropp

PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

Interim Report

EXPERIMENTAL STUDY OF THE DYNAMIC RESPONSE OF HIGHWAY BRIDGES

TO: J. F. McLaughlin, Director
Joint Highway Research Project

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

March 23, 1977

Project: C-36-56S

File: 7-4-19

The attached Interim Report is submitted on the HPR Part II Research Study titled "Bridge Vibration Studies". The Report is titled "Experimental Study of the Dynamic Response of Highway Bridges". It has been authored by Peter K. Kropp, Graduate Instructor in Research on our staff under the direction of Professors M. J. Gutzwiller and R. H. Lee.

This Interim Report covers the investigation of the basic response characteristics of typical highway bridges under passage of traffic. It will be followed by a Final Report on the Study early in the fall of 1977.

The Report is forwarded to the Board for acceptance as partial fulfillment of the objectives of the research. If accepted it will be forwarded to ISHC and FHWA for review, comment and similar acceptance in draft format prior to final publication and distribution. The Report attached does not include the extensive Appendices listed in the Table of Contents. Copies of the Appendices will be made available on request.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

HLM:ms

cc: W. L. Dolch
R. L. Eskew
G. D. Gibson
W. H. Goetz
M. J. Gutzwiller
G. K. Hallock
D. E. Hancher
M. L. Hayes
K. R. Hoover
G. A. Leonards
R. D. Miles
P. L. Owens
G. T. Satterly
C. F. Scholer
M. B. Scott
K. C. Sinha
L. E. Wood
E. J. Yoder
S. R. Yoder

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

http://www.archive.org/details/experimentalstud00krop

Interim Report

EXPERIMENTAL STUDY OF THE DYNAMIC RESPONSE OF HIGHWAY BRIDGES

by

Peter K. Kropp
Graduate Instructor in Research

Joint Highway Research Project

Project No.: C-36-56S

File No.: 7-4-19

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station
Purdue University

in cooperation with the

Indiana State Highway Commission

and the

U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views of policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, Indiana
March 23, 1977

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. JHRP-77-5	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle EXPERIMENTAL STUDY OF THE DYNAMIC RESPONSE OF HIGHWAY BRIDGES	5. Report Date March 23, 1977 6. Performing Organization Code
7. Author(s) Peter K. Kropp	8. Performing Organization Report No. JHRP-77-5
9. Performing Organization Name and Address Joint Highway Research Project Civil Engineering Building Purdue University West Lafayette, Indiana 47907	10. Work Unit No. 11. Contract or Grant No. HPR-1(14) Part II
12. Sponsoring Agency Name and Address Indiana State Highway Commission State Office Building 100 North Senate Avenue Indianapolis, Indiana 46204	13. Type of Report and Period Covered Interim Report 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Dept. of Transportation, Federal Highway Administration, Research Study titled "Bridge Vibration Studies".

16. Abstract

The objective of this investigation was to establish the basic response characteristics for common types of highway bridges under the passage of ordinary vehicular traffic. The response characteristics under consideration were the maximum values of displacement, velocity, acceleration, and jerk together with the frequency content and damping ratio. Sixty-two bridges of all types of construction with one through four spans were instrumented with a deflection gage and a varying number of accelerometers. More than 2200 vehicle crossings were recorded with the major portion being heavy trucks.

The results of the investigation indicate that the level of acceleration experienced by most bridges falls below the values which would be psychologically disturbing to pedestrians. The results also indicate that the fundamental torsional mode contributes as much or sometimes more to the total bridge motion as does the fundamental bending mode.

An algorithm is also presented which will permit the integration of acceleration in order to find velocity and/or displacement. Also a technique to differentiate the displacement or acceleration in order to find the corresponding velocity or jerk is presented.

17. Key Words Bridge Vibration; Bridge Design; Bridge Accelerations; Human Response to Bridge Vibration	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 99	22. Price

Form DOT F 1700.7 (8-69)

ACKNOWLEDGMENTS

The author wishes to express his appreciation to Professors R. H. Lee, M. J. Gutzwiller, C. D. Sutton, J. T. Gaunt, and R. E. Lynch, for their advice, assistance, and guidance throughout the course of this study.

For the collection of test data, the author wishes to thank Messrs. Barry Elkin, Paul L. Owens, and Fred Chuck, of the Indiana State Highway Commission Research and Training Center.

Special thanks is also extended to Professor Anshel J. Schiff for the use of the computer facilities in the Applied Stochastics Laboratory and to Mr. James K. Sprandel for his valuable assistance in digitizing the test data.

The financial assistance granted by the Joint Highway Research Project is gratefully acknowledged.

TABLE OF CONTENTS

Page

LIST OF TABLES. vii

LIST OF FIGURES ix

LIST OF SYMBOLS xvi

HIGHLIGHT SUMMARY xix

CHAPTER I - INTRODUCTION. 1

- 1.1 General Objectives. 1
- 1.2 Scope of this Investigation 3

CHAPTER II - DESCRIPTION OF THE TEST PROGRAM. . . . 5

- 2.1 General 5
- 2.2 Bridge Selection Procedures 6
- 2.3 Test Variables. 9
  - 2.3.1 Test Vehicle 10
    - Test Vehicle Parameters. 12
- 2.4 Instrumentation and Testing Procedures. . 12
  - 2.4.1 General. 12
  - 2.4.2 Test Apparatus 17
  - 2.4.3 Instrumentation. 18
    - Phase II Instrumentation 18
    - Phase I Instrumentation. 22
  - 2.4.4 Test Procedures. 25
    - Phase I Test Procedures. 28
    - Phase II Test Procedures 29
    - Required Conditions for Vehicle Crossing to be Collected. . . . 30
    - Test Vehicle (Research and Training Center Bus) 31
  - 2.4.5 Special Tests. 32

CHAPTER III - REDUCTION OF THE TEST DATA. 33

- 3.1 General 33
  - 3.1.1 Data Classification. 33
- 3.2 Digitization of Data. 34
  - 3.2.1 Digitization Procedures. 35

TABLE OF CONTENTS (Continued)

Page

3.3 Analysis of the Test Data 38
  3.3.1 Computer Programs Used in Data Analysis 40
  3.3.2 Frequency Content of the Data. . . 54
    Frequency Response Analysis. 54
    Frequency Spectrum Computer Programs. 56

CHAPTER IV - SUMMARY OF TEST RESULTS. 57

4.1 General 57
4.2 Analysis Procedures 57
4.3 Results of the Testing Program. 61
4.4 Results of the Frequency Analysis 80

CHAPTER V - SUMMARY AND CONCLUSIONS 90

5.1 Summary of Test and Analysis Procedures . 90
5.2 Summary of Results and Conclusions. . . . 92

BIBLIOGRAPHY. 96

APPENDICES

Appendix A - List of Bridge Structures Tested. 100
Appendix B - Test Vehicle Parameters 107
Appendix C - Test Apparatus. 111
  C.1 Deflection Gage. 111
Appendix D - Special Test Procedures 119
Appendix E - Digitization of the Test Data . . 122
  E.1 Equipment. 122
    E.1.1 Data Acquisition System (DAS) 122
    E.1.2 Mini-Computer System. 122
      Mainframe 122
      Disk. 123
      CRT Terminal. 123
      Magnetic Tape 123
      Line Printer. 123
  E.2 Details and Explanation of Digitization Procedures. 123
    E.2.1 Quantizing Levels and Sampling Interval. 124
    E.2.2 Computer Programs 126
Appendix F - Computer Program and Techniques Used to Analyze the Data. 153
  F.1 General. 153
  F.2 Computer Program Documentation . . . 153
  F.3 Differentiation Techniques 154
  F.4 Integration Techniques 154

TABLE OF CONTENTS (Continued)

Page

F.4.1 Background. 154
F.4.2 Computation of Displacement and Velocity from Acceleration. . . 156
Single Span Bridges 158
Two Span Bridges. 160
Three and Four Span Bridges . 162
Appendix G - Frequency Response Analysis . . . 205
G.1 General. 205
G.2 Spectral Analysis. 205
G.2.1 Computing the Fourier Spectrum. 206
G.2.2 Additional Uses of the Fourier Spectrum. 213
G.2.3 Example of a Fourier Spectrum 215
G.3 Computer Program Used to Calculate the Fourier Spectrum 217
Appendix H - Fourier Spectra of the Test Data. 230

LIST OF TABLES

Table		Page
2.1	Ten Categories of Bridges Selected for the Testing Program.	7
2.2	Vehicle Classification Codes for Phase I and II Testing	11
2.3	Summary of Test Vehicle Parameters	14
2.4	Summary of Tape Recorder Hook-up to Accelerometers and Deflection Gage for all Bridges Tested	21
4.1	Most Efficient Smoothing Used in Differentiating the Displacement and Acceleration to Calculate the Corresponding Acceleration and Jerk	60
4.2	Summary of the Results of the Testing Program	62
4.3	Relationship Between the Vehicle Weight Factor and the Vehicle Classification Code .	66
4.4	Summary of the Results of the Testing Program for Vehicles with a Weight Factor of 9.	68
4.5	Summary of the Mean Values from Table 4.2 for Steel Beam or Plate Girder Bridges and One Through Three Spans.	71
4.6	Acceleration Criteria for Human Response to Harmonic Vertical Vibration	78
4.7	Typical Damping Ratios	79
4.8	Summary of the Frequency Analysis.	81
4.9	Identification of the Fundamental Torsional Frequency by Comparison of the Percentage of the Total Mean Square Contribution for Each Frequency Excited.	86

LIST OF TABLES (Continued)

Appendix Table | Page

C.1 First Three Natural Frequencies of the Deflection Gage Wire 113

E.1 Phase I Digitization Computer Program. . . . 130

E.2 Phase II Digitization Computer Program . . . 140

E.3 Description of Magnetic Tapes Generated from the Digitized Test Data. 148

E.4 Data Format on Both the 7 and 9-Track Magnetic Tapes. 149

F.1 Computer Program Used for Reduction of Phase II Data. 167

G.1 Summary of Fourier Spectrum for the Example Problem. 219

G.2 Computer Program Used to Calculate the Fourier Spectrum for Phase II Data 221

LIST OF FIGURES

Figure | Page

2.1 Test Vehicle - Research and Training Center Bus 13

2.2 Accelerometer and Deflection Gage Location for Phase I and II Testing 20

2.3 Portion of an Oscillographic Strip Chart Showing the Outputs of Channels 1, 2, 3, and 4 for a Vehicle Crossing a Two Span Bridge--Phase II 23

2.4 Schematic of Phase II Instrumentation for a Two Span Bridge. 24

2.5 Portion of an Oscillographic Strip Chart Showing the Outputs of Channel 1, 3, and 4 for a Vehicle Crossing a Three Span Bridge--Phase I. 26

3.1 Equipment Set-up for Digitizing the Data . . 36

3.2 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 41

3.3 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 42

3.4 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 43

3.5 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 44

3.6 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 45

3.7 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 46

3.8 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . 47

LIST OF FIGURES (Continued)

Figure

3.9 Comparison of Twice Integrated Acceleration Record with the Actual Deflection Record . . . 48

3.10 Comparison of Twice Differentiated Deflection Record with the Actual Acceleration Record . . 49

3.11 Comparison of Twice Differentiated Deflection Record with the Actual Acceleration Record . . 50

3.12 Comparison of Twice Differentiated Deflection Record with the Actual Acceleration Record . . 51

3.13 Comparison of Twice Differentiated Deflection Record with the Actual Acceleration Record . . 52

4.1 Maximum Displacement versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus. 73

4.2 Maximum Bridge Velocity versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus. 74

4.3 Maximum Acceleration versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus 75

4.4 Maximum Jerk versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus. 76

Appendix
Figure

B.1 Load-Deflection Curves for Test Vehicle Springs. 109

B.2 Load-Deflection Curves for Test Vehicle Tires.110

C.1 Research and Training Center Mobile Lab and Tow Vehicle. 114

C.2 Interior of the Mobile Lab Showing the Test Apparatus Including Three Tape Recorders and Switching Box. 115

LIST OF FIGURES (Continued)

Appendix
Figure

C.3 Typical Accelerometer Installation on the Bridge Curb 116

C.4 Typical Deflection Gage Installation. . . . 117

C.5 Deflection Gage Installation Showing Instrumented Cantilevered Aluminum Beam 118

E.1 Flowchart for the Phase I Digitization Computer Program. 128

E.2 Flowchart for the Phase II Digitization Computer Program. 129

F.1 Flowchart for the Phase II Data Reduction Computer Program. 164

F.2 Additional Correction Applied to the Calculated Deflection for Two Span Bridges . . . 166

G.1 1/10th Cosine Taper Data Window 212

G.2 Illustration of the Boxcar Function, $u_b(t)$. 212

G.3 Fourier Spectrum of the Time Series Whose Waveform is Composed of 3Hz, 9Hz, and 15Hz Sine Waves. 218

G.4 Flowchart for the Computer Program Used for Calculating the Fourier Spectrum. 220

H.1a Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Bridge Centerline 230

H.1b Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Travel Lane 231

H.1c Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Curb Lane 232

H.2a Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Bridge Centerline 233

LIST OF FIGURES (Continued)

Appendix Figure | Page

H.2b Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Travel Lane 234

H.2c Fourier Spectrum of the Free Vibration of Acceleration, Transverse Vehicle Position-Curb Lane 235

H.3a Fourier Spectrum of the Free Vibration of Deflection, Transverse Vehicle Position-Bridge Centerline 236

H.3b Fourier Spectrum of the Free Vibration of Deflection, Transverse Vehicle Position-Travel Lane 237

H.3c Fourier Spectrum of the Free Vibration of Deflection, Transverse Vehicle Position-Curb Lane 238

H.4a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 239

H.4b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 240

H.4c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 241

H.5a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 242

H.5b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 243

H.5c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 244

H.6a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 245

H.6b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 246

H.7a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 247

LIST OF FIGURES (Continued)

Appendix Figure — Page

H.7b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 248

H.7c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 249

H.8a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 250

H.8b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 251

H.9a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 252

H.9b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 253

H.9c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 254

H.10a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 255

H.10b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 256

H.11a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 257

H.11b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 258

H.11c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 259

H.12a Fourier Spectrum of the Free Vibraiton Portion of the Deflection Record 260

H.12b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 261

H.13a Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 262

H.13b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 263

LIST OF FIGURES (Continued)

Appendix Figure — Page

H.14a Fourier Spectrum of the Vehicle Crossing Portion of the Deflection Record. 264

H.14b Fourier Spectrum of the Free Vibration Portion of the Deflection Record 265

H.14c Fourier Spectrum of the Vehicle Crossing Portion of Accelerometer No. 1. 266

H.14d Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 267

H.14e Fourier Spectrum of the Vehicle Crossing Portion of Accelerometer No. 2. 268

H.14f Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 269

H.15a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 270

H.15b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 271

H.16a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 272

H.16b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 273

H.16c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 274

H.17a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 275

H.17b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 276

H.17c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 277

H.18a Fourier Spectrum of the Free Vibration Portion of the Delfection Record 278

H.18b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 279

LIST OF FIGURES (Continued)

Appendix
Figure   Page

H.18c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 280

H.19a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 281

H.19b Fourier Spectrum of the Free Vibration Portion Accelerometer No. 2. 282

H.20a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 283

H.20b Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 1 284

H.20c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 285

H.21a Fourier Spectrum of the Free Vibration Portion of the Deflection Record 286

H.21b Fourier Seectrum of the Free Vibration Portion of Accelerometer No. 1 287

H.21c Fourier Spectrum of the Free Vibration Portion of Accelerometer No. 2 288

LIST OF SYMBOLS

a_n	digitized acceleration file
a_n^*	corrected digitized acceleration file
A	amplitude of a signal corresponding to the maximum quantizing level
b	total width of bridge deck
b_i	constants associated with the baseline correction for acceleration
C_k	real portion of the discrete Fourier transform
EI	stiffness of the steel bridge beams
E_c	modulus of elasticity of concrete
E_k	correction made to the twice integrated acceleration to obtain the final displacement file
FFT	Fast Fourier Transform
f_b^1	fundamental bending frequency
f_c	Nyquist frequency or sampling rate in samples/second
f_i	ith natural frequency of any vibrating system
f_t^1	fundamental torsional frequency
$f_{t,i}$	pseudo frequency of the ith axle if only the tire springs acts
$t_{ts,i}$	pseudo frequency of the ith axle if both the tire and suspension springs act in series
g	gravitational acceleration (386 in/sec^2)
G(f)	power spectral density function in units^2/Hz
G_k	discrete power spectral density function in units^2/Hz

j	imaginary number $(-1)^{1/2}$
J	degree of smoothing polynomial used in differentiating either displacement or acceleration
k	integer constant
K	number of data points used in conjunction with the smoothing polynomial, J
$k_{s,i}$	stiffness of the ith suspension spring acting alone
$k_{t,i}$	effective stiffness of the tires for the ith axle
$k_{ts,i}$	effective stiffness for the ith axle when the suspension spring and tire spring act in series
K_{slab}	stiffness of the bridge deck
L	length of deflection gage wire
m	mass concentrated at a point
n	integer constant
N	constant denoting the number of samples in a file
PSD	power spectral density
$P_{st,i}$	static reaction at the ith axle
Q_k	imaginary portion of the discrete Fourier transform
$Q(b_k)$	used in minimization of the $(v_n-\bar{v}_n)^2$
t	thickness of bridge deck slab
t	time, independent variable of time history
T	tensile force in a wire
T	record or file length in seconds $(=N\Delta t)$
$U_c(t)$	1/10th cosine taper data window
$U_b(t)$	boxcar window function
v_n	raw digitized velocity file obtained from integrating a_n
v_n^*	corrected digitized velocity file

$\bar{v}_n$	curve to which the velocity file (v_n) is to be least square fit
x_n	discrete time series
$x(t)$	response time history
X_k	discrete Fourier transform of x_n
$\|X_k\|$	modulus of the Fourier transform, X_k
$X(f)$	Fourier transform of $x(t)$
$X(f,T)$	estimate of the Fourier transform of $x(t)$
y_n	discrete displacement file
θ_k	phase angle associated with the Fourier transform X_k
ε_n	error introduced during recording and digitization
Δf_s	discrete sampling frequency
Δt	time interval or sample spacing (seconds)
Δx	increment between successive quantizing levels
μ	Poisson's ratio
ψ_x^2	mean square value of x
$\bar{\psi}_x^2$	estimate of the mean square value of x.
$\bar{\psi}_{fn}^2$	estimate of the mean square value of a certain discrete frequency content
$\bar{\psi}_{fm}^2$	mean square value of x for a particular frequency, f_m
ψ_t^2	mean square value of the 1/10th cosine taper window, $U_c(t)$
ω	frequency in radians/second

HIGHLIGHT SUMMARY

The objective of this investigation was to establish the basic response characteristics for common types of highway bridges under the passage of ordinary vehicular traffic. The response characteristics under consideration were the maximum values of displacement, velocity, acceleration, and jerk together with the frequency content and damping ratio. Sixty-two bridges of all types of construction with one through four spans were instrumented with a deflection gage and a varying number of accelerometers. More than 2200 vehicle crossings were recorded with the major portion being heavy trucks.

The results of the investigation indicate that the level of acceleration experienced by most bridges falls below the values which would be psychologically disturbing to pedestrians. The results also indicate that the fundamental torsional mode contributes as much or sometimes more to the total bridge motion as does the fundamental bending mode.

An algorithm is also presented which will permit the integration of acceleration in order to find velocity and/or displacement. Also a technique to differentiate the displacement or acceleration in order to find the corresponding velocity or jerk is presented.

CHAPTER 1: INTRODUCTION

1.1. General Objectives

Although the theoretical dynamic response of single-span and multi-span highway bridges under moving vehicles has been the subject of numerous investigations in recent years(16-34), the comprehensive study of the actual dynamic performance of a wide range of bridge types has had little attention.

The objective of this study is to provide information which will provide a better understanding of the dynamic performance of a broad range of highway bridge structures under the passage of ordinary vehicular traffic and to correlate this information with the predicted or theoretical performance. The ultimate goal of the investigation is to provide a better understanding of bridge vibrations, leading to the establishment of design criteria for inclusion in the bridge specifications which will directly regulate the dynamic response of a bridge structure.

Design practice, as outlined in the present bridge design codes (35), avoids the direct treatment of the dynamic behavior of bridge-vehicle systems. Restrictions are imposed upon the static deflection-span ratio under a stationary live load and upon the girder depth-span ratios with the hope that the restrictions will provide for a satisfactory dynamic performance.

These restrictions have no rational basis in terms of dynamic performance and the results from observing these rules seems to substantiate this fact.

One of the benefits which could be realized from code requirements which successfully predict dynamic performance would be the exclusion of bridges before they were built, which would have adverse psychological effects upon pedestrians or upon occupants in vehicles crossing the bridge. A recent paper by Shahabadi (1) presents a summary of previous human response studies to vertical vibrations and relates these studies to the particular problem of human response to bridge motion. Another benefit which could be realized would be the increased use of high strength steel for bridge girders. This would be possible by relaxing the present limitations on deflections and/or span-depth ratios by showing that increasing the flexibility of a bridge would not result in unsatisfactory dynamic performance. The economical use of high-strength steel girders would result in significant savings if the design of these girders could be freed from arbitrary limitations which do not guarantee improvement in bridge performance.

In view of the general nature of the problem and circumstances outlined in this section, it is apparent that a method of analysis which will determine the dynamic response of highway bridges is required. The most successful studies in recent years have been presented by Oran(20) for simple span highway bridges and by Veletsos and Huang(36,19) for multi-span continuous bridges. The theory was verified by comparisons with the results of laboratory studies on simply supported beams and with results of the AASHO Road Test bridge studies(37), although the bridges tested in the AASHO study were single lane, simple span structures conforming rather closely to the assumptions of the theory. Later Aramaks(2) modified the analytical computer programs of Oran(20) and Huang(19) and conducted an acceleration study of highway bridges using

parameters related to the bridge, to the vehicle and the initial conditions of both the vehicle and the bridge. The effects of the major parameters on the bridge accelerations were studied and compared to an acceleration criteria for human response presented by Wright and Walker(26).

This investigation was conducted to verify the theoretical results presented by Oran(20), Huang(19) and Aramraks(2) in addition to providing a better understanding of the dynamic performance of highway bridges under ordinary traffic.

1.2 Scope of this Investigation

The investigation reported herein can be conveniently divided into the following three parts:

(a) Testing of a broad range of bridge structures under actual traffic conditions, collecting deflection and acceleration data.

(b) Reduction and analysis of the collected data to establish basic response characteristics as a function of structure parameters.

(c) Comparison between the analytical and experimental results.

In this investigation a sample of 62 bridges of various span lengths and various types of construction were instrumented and the deflection and acceleration response to highway traffic were recorded. In the second phase of testing, three representative structures were chosen from the original 62 bridges, and detailed tests were conducted. This phase of testing involved running a standard test vehicle at different longitudinal locations and at various speeds across the bridges and collecting the corresponding deflection and acceleration response. In addition, the deflection and acceleration transducers were relocated to different positions on the bridge to prove or

disprove results obtained using the analytical program. Also, a detailed profile was run on each of these three bridges since it has been shown that roughness of the bridge deck and approaches is one of the most important parameters affecting the dynamic motion(2). The field test data was then digitized and stored on magnetic tape for further analysis. In general, this analysis consisted of finding the maximum deflection, velocity, acceleration and jerk for each vehicle crossing and also involved analyzing the frequency content of each record. These results will then be compared with the results from the analytical program by Shahabadi(1).

In order to make this study as general as possible, the scope of the test program did not include obtaining specific vehicle parameters for each vehicle crossing a particular bridge. Only the vehicle's velocity and general type are known, except in the case where the standard test vehicle was used. Therefore, vehicle parameters such as tire and spring stiffness required for input into the analytical program were taken from manufacturers' data and test reports(19,37,38). However, the test results alone provide valuable information about the dynamic behavior of bridges under actual traffic conditions. Also, it was realized that all of the data which was collected could not be reduced and thoroughly interpreted within the time limits of this investigation. This investigation was designed to provide, after the completion of the present study, information valuable for further work in this field.

CHAPTER II: DESCRIPTION OF THE TEST PROGRAM

2.1. General

The purpose of this chapter is to describe the test program, the bridge selection procedure, and to briefly discuss the instrumentation and the data collection procedures. A more detailed description of the instrumentation and data collection procedure may be found in the appropriate parts of Ref. 3.

The testing program consisted of obtaining the dynamic response information from a group of 62 bridge structures selected for testing. This dynamic response information was in the form of deflection and acceleration records recorded on ¼" magnetic tape at each bridge site. The data was collected in two phases. The first phase was conducted during the summer of 1973 and included the testing of all 62 bridges. Due to instrumentation problems which occurred during the first phase, a second phase of collection was conducted during the summer of 1976 on 18 of the original 62 bridges. This second phase of testing also included testing of a few new bridges not included in the original sample of sixty-two. Their inclusion in the testing program was due to the fact that they carried high volumes of traffic, were readily accessible for testing and were physically compatible with the input for the analytical programs. Hereafter the first phase and second phase of testing will be referred to as Phase I and Phase II testing, respectively.

2.2. Bridge Selection Procedure

The types of bridges to be tested were selected so as to provide a representative sample of the bridges commonly in use today and were taken from a listing of all bridges on state and interstate highways in Indiana. The first step in this process was the selection of categories of bridges based upon their type of construction and the current popularity of this type of construction. This would immediately exclude such types of construction as stone arch and steel pony arch. Another factor considered in the selection of bridge categories was the number of bridges which had similar physical dimensions and were of the same type of construction. It was felt that this limitation was necessary due to the fact that one of the objectives of the test program was to investigate the premise that bridges of similar construction and physical dimension would have similar dynamic response under highway traffic. Ten categories of bridges were selected and are shown in Table 2.1.

The ten categories were then broken down into sub-categories with each sub-category containing bridges having the same number of spans, the same span lengths, identical deck width, and, wherever possible, all bridges in a sub-category were constructed within the same year. In some sub-categories the range of time when a bridge was built was two or three years and in these cases all bridges in the sub-category were designed using the same design standards.

Originally it was attempted to use additional selection factors for bridges in sub-categories, such as skew and average daily traffic load, but it was practically impossible to find bridges which were identical in these respects along with the four factors mentioned in the previous paragraph.

Table 2.1. Ten Categories of Bridges Selected for the Testing Program

Category Number	Type of Construction
1	Non-Composite Steel Beam (Single-Span)
2	Non-Composite Continuous Steel Beam
3	Composite Non-Continuous Steel Beam
4	Composite Continuous Steel Beam
5	Composite Continuous Plate Girder (essentially uniform moment of inertia)
6	Composite Continuous Plate Girder (non-uniform moment of inertia)
7	Non-Continuous Reinforced Concrete Girder
8	Continuous Reinforced Concrete Girder
9	Continuous Reinforced Concrete Slab
10	Prestressed Concrete I-Beam Bridge

The final factor used in the selection of sub-categories was the type of highway the bridge carried. Highways were classified into the following groups:

(1) two lane county roads with two-directional traffic crossing over Interstate Highways
(2) two lane state highways with two-directional traffic
(3) four lane state highways
(4) four lane divided Interstate highways
(5) six lane divided Interstate highways

In many cases the bridges located on four or six lane divided highways were twin structures and were considered as two separate bridges. The majority of bridges selected for testing were of the steel beam or plate girder type. This was done since one of the objectives of the study was to investigate the use of high strength steel beams in bridge construction. Additional factors influencing the selection of sub-categories were the total number of spans and the section properties of the bridge cross-section since the analytical programs are restricted to bridges having constant moment of inertia, equal spans for two span bridges, and equal outer spans for three-span structures.

The end result of the selection process was a sample of 62 bridges consisting of 40 steel span structures and 22 concrete span structures. The complete listing of the 62 bridges including category, sub-category, span length, width, and year built can be found in Appendix A. The ten categories which were based on type of construction are divided into 22 sub-categories with two to four sub-categories per category. The sub-categories consist of one to six similar bridges based on number of spans, span length, deck width, and year built. These four selection factors varied widely between sub-categories with the

number of spans varying from one to four, the span lengths from 27.0 feet to 129.0 feet, the deck width from 24.0 feet to 51.0 feet and the year built from 1929 to 1972.

In conclusion, the sample of bridges selected contains a diverse variety of bridge types, sizes, ages and traffic conditions. The sample contains a selection of small groups of essentially identical bridges which will make it possible to identify similar dynamic properties. The whole sample of 62 bridges will make it possible to identify general dynamic properties such as maximum accelerations or fundamental bending frequencies.

2.3. Test Variables

Some of the test variables, none of which were controlled, were:

(1) speed of the vehicle crossing the bridge
(2) total weight of the vehicle
(3) type of vehicle
(4) vehicle parameters such as spring stiffness and natural frequency of its system of tires and springs
(5) transverse location of the vehicle upon the bridge
(6) initial conditions of the vehicle as the vehicle enters the bridge

During each test, the test equipment and personnel were in such a position so as to minimize any effects their presence would have on normal traffic flow. Only during the latter portion of the Phase II testing were the testing crew required to place a warning sign up the road from the bridge which read "Road Work Ahead". From the observations of the testing personnel, the sign had little or no effect on the traffic's velocity or transverse position on the bridge.

Two different types of vehicle coding were used, one type for the first phase of testing where all 62 bridges were tested and another type for the second phase of testing during the summer of 1976. See Table 2.2 for the Phase I and Phase II vehicle classification codes. The major difference between the two codes is the consolidation of all truck types into one category for the first phase of testing whereas for the second phase there are six categories ranging from a single tandem drive axle unit to a tandem drive axle tractor with a two axle trailer. During Phase I testing, one of the four vehicle codes were recorded as a signal pulse on the ¼" magnetic tape as each vehicle traversed the bridge and, if the vehicle were a truck, a test crew member would record a general description of the truck on a log sheet. Since the general description did not include the number of axles and also since the transfer of information from the log sheet to digitized data storage tapes greatly slowed the reduction process, it was decided that for the second phase of testing no log sheets would be used and the vehicle codes expanded to nine categories instead of the original four.

Expanding the vehicle code categories had another advantage wherein the use of existing loadometer studies could make the estimation of vehicle length and range of axle weights more reliable. Since the vehicle axle weights and spacings were unknown, an estimation of these quantities had to be made for input to the analytical programs.

2.3.1. Test Vehicle

Even though one of the main objectives of the test program was to survey a broad range of bridge structures under normal traffic loadings it was felt that a test vehicle of some sort should be used, where vehicle

Table 2.2. Vehicle Classification Codes for Phase I and II Testing

Code	Vehicle Description
Phase I	
1	Passenger Car
2	Pick-ups to 2-ton trucks
3	Tractor-trailers, Dumptruck, etc.
4	Test Vehicle (Research & Training Center Bus)
Phase II	
1	Passenger Car
2	Test Vehicle (R & TC Bus)
3	Pick-up/light delivery vans
4	Two axle truck
5	Three axle truck
6	Two axle tractor-one axle trailer
7	Two axle tractor-two axle trailer
8	Three axle tractor-one axle trailer
9	Three axle tractor-two axle trailer

parameters such as axle spacing, axle weight, and spring stiffness could be measured. Having these quantities to input to the analytical programs would provide more confidence when comparing the analytic results with the actual results. In areas of low traffic volume, such as a county road over an interstate, a vehicle of sufficient weight to give adequate excitation to the bridge structure was needed. This requirement was another reason for the use of a test vehicle.

The test vehicle for this testing program was a school bus owned by the Indiana State Highway Commission's Research and Training Center and is shown in Figure 2.1.

Test Vehicle Parameters

The bus is a 1969 International bus with dual rear wheels, a wheel base of 23.0 feet and was loaded with broken concrete test beams to achieve significant axle loads. The vehicle parameters which were measured were axle spacing, front and rear axle weights, front and rear spring stiffness and front and rear tire stiffness. The testing procedure and details used to obtain these parameters may be found in Appendix B. A summary of test vehicle parameters is shown in Table 2.3.

2.4. Instrumentation and Testing Procedure

2.4.1. General

This section will contain a discussion of the data collection process used to obtain the dynamic response information from the 62 bridge structures selected for testing. As was previously mentioned, the dynamic response information was in the form of deflection and acceleration records recorded on ¼" instrumentation magnetic tape at each bridge site and later digitized and

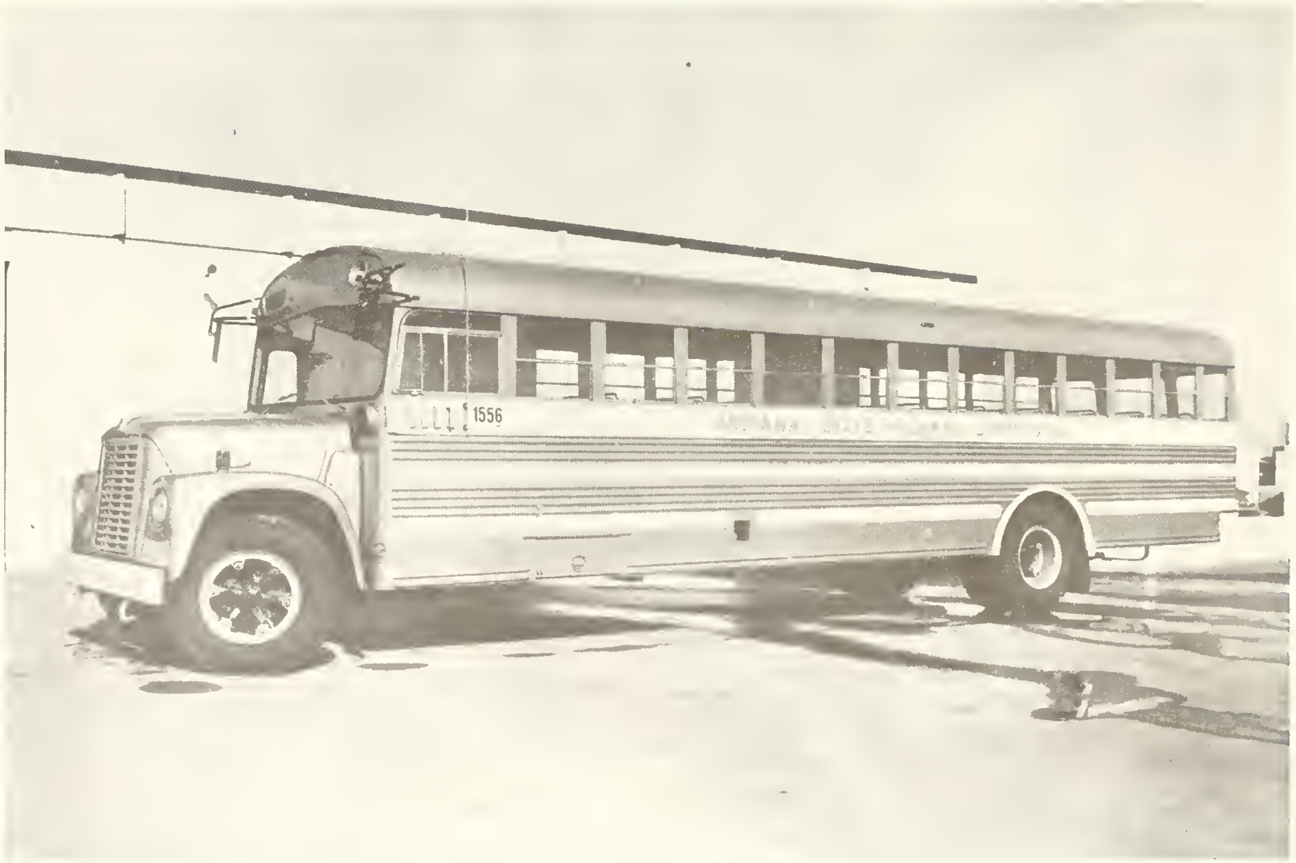


Figure 2.1. Test Vehicle - Research and Training Center Bus

Table 2.3. Summary of Test Vehicle Parameters*

Vehicle Type: 1969 International School Bus
23.0 ft. wheel base with dual rear wheels

Axle Weights:		
	Front Axle	6760 lbs.
	Rear Axle	14240 lbs.
	Gross Vehicle Weight	21000 lbs.

Spring Constants (lbs/in):

	Front	Rear
Tires, k_t	6153.	13668.
Springs, k_s	3391.	6273.

Pseudo Frequencies (cps):

	Front	Rear
If only tire spring acts, $f_{t,i}$	2.98	3.06
If tire and suspension system act in series $f_{ts,i}$	1.78	1.72

*Note. The values given are for Phase II testing. The values for Phase I are identical for all practical purposes.

stored on 2450 foot, ½" magnetic tape. The digitization phase of the project will be discussed in further detail in the next chapter.

Previous investigations of the behavior of bridges under dynamic load(16,24) have employed testing methods which result in deformation or strain data. In this investigation acceleration and jerk data were of greater interest since these quantities have more significance in the area of human response to motion than does deflection(2). For this reason it was decided to collect acceleration data using accelerometers positioned on the bridge so as to record the maximum dynamic effect for each vehicle crossing. Since studies to date(16,24) have shown that highway bridges vibrate primarily in their fundamental mode, two accelerometers were placed at midspan of each span, one on either side of the bridge, and attached to the curbs. Accelerometers were placed in each span since the maximum acceleration for a vehicle crossing would not necessarily be a maximum in any one particular span. One accelerometer on each side of the bridge in each span was used so that it would be possible to study the torsional characteristics of each bridge tested.

In order to make the study as general as possible, the deflection at one point on the bridge was also collected. The deflection gage was positioned adjacent to an accelerometer so that the following operations could be performed: (1) An instrumentation check could be performed if the deflection record were differentiated twice and compared with the actual acceleration record, (2) Intergration routines could be developed which would allow placing accelerometers on a bridge, and by integrating the acceleration records twice, the deflection record could be obtained. This would be desirable in the case where physical conditions under the bridge would

not permit the use of a deflection gage and either velocity or deflection were required, (3) Finally, since jerk transducers were not used, a check had to be provided for the case where the acceleration record was differentiated to find the corresponding jerk record. This check was made by differentiating the deflection record three times using certain smoothing polynomials which would give a range of jerk values. This range of values could then be compared with results obtained by differentiating the corresponding acceleration file once.

Since this testing program would collect a substantial amount of data, more efficient and accurate ways to reduce this data had to be found. In previous investigations(16,24) the output from the instrumentation was recorded either by oscillographic equipment or mechanically recorded on a strip chart. In either case the data had to be reduced by visual or mechanical manipulation of the output from the strip chart. It was therefore decided to use instrumentation tape recorders and store the data from the accelerometers and deflection gage in analog form on magnetic tape. The data could subsequently be digitized using an analog-to digital converter and be stored in digital form on magnetic tape. Having the data in digital form would make it possible to reduce the data using high-speed digital computers, thus eliminating the errors which are inherent in reducing data from a strip chart. The analog-to-digital converter and associated software were provided by Professor Ansel Schiff through his Applied Stochastics Laboratory and the data was reduced on the Dual CDC 6500 Computer System operated by the Purdue University Computer Center.

The remainder of this chapter will be devoted to the description of apparatus and instrumentation used and the procedures followed for the testing program.

2.4.2. Test Apparatus

The following section will contain a brief description of the major pieces of equipment used to collect and store the data, and, will include the accelerometers, deflection gage and tape recorders. For additional information concerning these instruments see Reference (3).

Kistler Model QA-116-15 accelerometers were used to measure the acceleration of certain points (usually midspans) of the structure as vehicles traversed the bridge. This accelerometer is a self-contained instrument for measuring steady state and moderate frequency vibrations with the capability of sensing accelerations up to 15 g's over a wide range of temperatures and environmental conditions.

The deflection gage consisted of a cantilever aluminum plate mounted on a block of concrete with strain gages attached to the cantilevered plate. The free end of this plate was connected to the bridge structure by a tensioned wire so as the bridge deflected so would the cantilever beam. As the cantilevered beam deflected, a corresponding strain would be produced in the strain gages which, in turn, produced an electrical output corresponding to the particular displacement.

Tandberg Series 100 Instrumentation Tape Recorders were used to record the outputs from both the accelerometers and deflection gages together with information concerning the vehicle type and the position of the vehicle on the bridge. Each recorder has the capacity to simultaneously collect four separate inputs and record these inputs on ¼" x 1800 ft. magnetic tape at speeds of 1 7/8, 3 3/4 or 7½ inches per second.

2.4.3. Instrumentation

In this section the instrumentation of the 62 bridges selected for testing will be discussed. In some respects instrumentation for Phase I and Phase II data collection was different; therefore, the instrumentation for Phase II will be discussed first followed by Phase I instrumentation.

Phase II Instrumentation (for data collected during 1976)

In keeping with the objectives of this test program each bridge tested was instrumented with one deflection gage and from two to eight accelerometers depending upon the number of spans. Phase II instrumentation permitted the collection records for vehicles moving in only one direction across the bridge which was not the case for Phase I.

For most tests, two accelerometers were placed at midspan of each span, one on either side of the bridge and attached to the curbs. For the purpose of consistency throughout this paper, the accelerometers will be numbered from one through eight with Accelerometer #1 being in the first span on the right hand curb and Accelerometer #2 on the left curb. The right and left hand directions are taken as seen by a driver of a vehicle entering the first span. If required, Accelerometer #3 was positioned at midspan of the second span on the right curb and Accelerometer #4 was on the left curb. The deflection gage was always positioned adjacent to Accelerometer #1 for reasons previously discussed in Section 2.4.1. There were some cases where the deflection gage could not be positioned at midspan of the first span due to high water under the bridge. For these cases the deflection gage was placed as close as possible to midspan and the two accelerometers would be moved from their usually midspan

position to the same longitudinal position as that for the deflection gage. See Figure 2.2 for transducer locations for bridges having from one to four spans.

The output signals from the accelerometers and deflection gage were recorded at 3 3/4 ips on ¼" x 1800' magnetic tape using from one to three 4-channel tape recorders, depending on the number of accelerometers used. One tape recorder was sufficient for a single span bridge instrumented with two accelerometers and one deflection gage. For a four span bridge, eight accelerometers and one deflection gage would be used which would require 3 tape recorders. See Table 2.4 for the number of tape recorders and the use of their corresponding channels for one through four span bridges.

Channel 4 of each tape recorder was reserved for information concerning the type of vehicle crossing the bridge and the position of this vehicle with respect to the bridge (i.e. Is the vehicle approaching, on, or off the bridge?). The type of vehicle crossing the bridge was recorded as a certain voltage level of a certain time duration by a digital vehicle code entrant device. Signals from air-activated switches coupled to traffic hoses were used to indicate the longitudinal position of the vehicle with respect to the bridge. Since the air-activated switches were common to Channel 4 of each tape recorder, the signals from these switches provided a common time base between the tapes being recorded. Three traffic hoses are used. The first hose is attached to the roadway surface approximately 100 feet before the beginning of the bridge with the second hose attached to the approach slab adjacent to the expansion joint. The third and final hose is fastened to the approach slab adjacent to the last expansion joint.

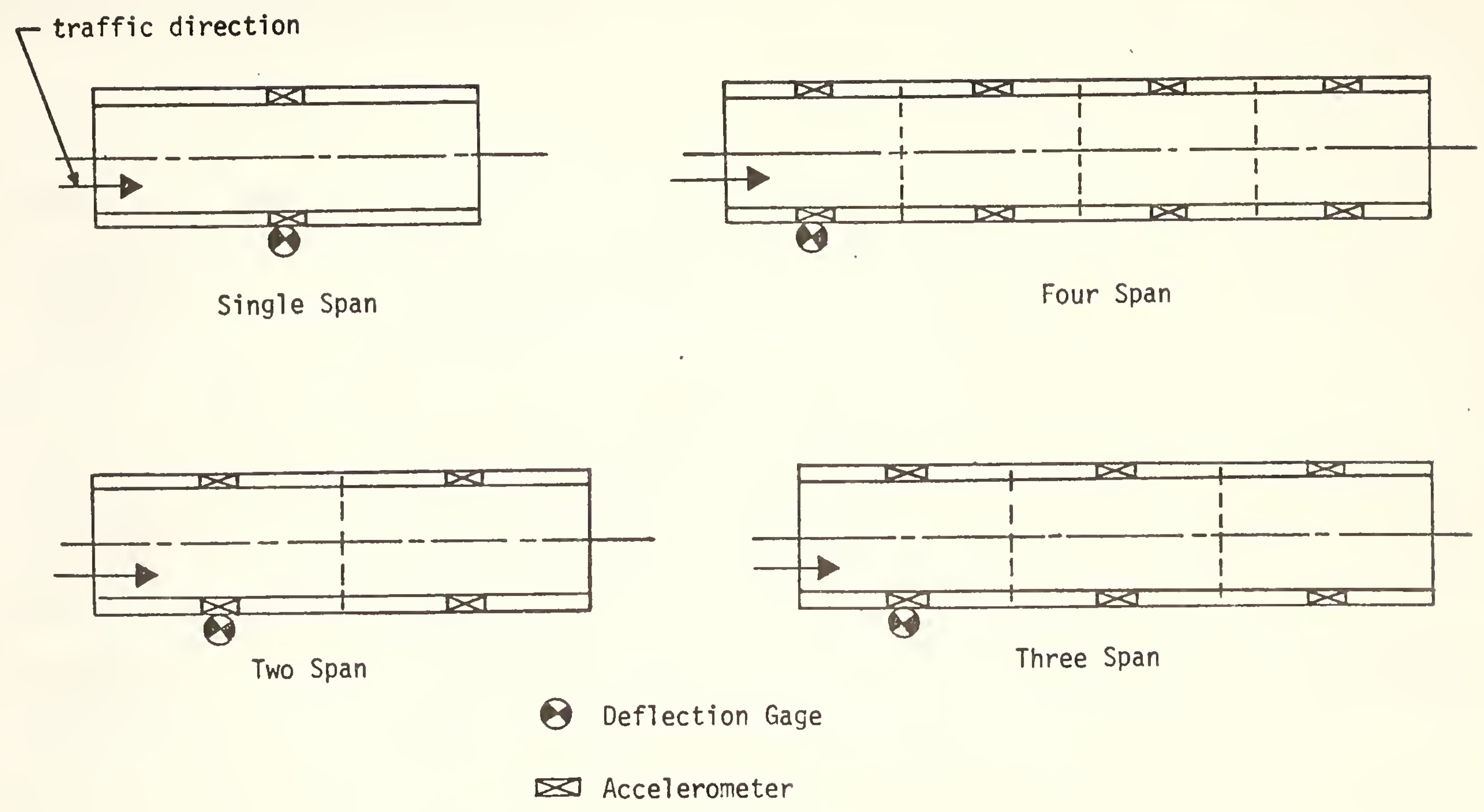


Figure 2.2. Accelerometer and delfection gage location for Phase I and II testing.

Table 2.4. Summary of Tape Recorder Hook-up to Accelerometers and Deflection Gage for All Bridges Tested

		Tape Recorder	
	#1	#2	#3
Single Span Bridge			
Channel 1	Deflection Gage	Not Used	Not Used
Channel 2	Accelerometer #1		
Channel 3	Accelerometer #2		
Channel 4	Information		
Two Span Bridge			
Channel 1	Deflection Gage	Accelerometer #3	Not Used
Channel 2	Accelerometer #1	Accelerometer #4	
Channel 3	Accelerometer #2	Free	
Channel 4	----------Information------------		
Three Span Bridge			
Channel 1	Deflection Gage	Accelerometer #3	Accelerometer #5
Channel 2	Accelerometer #1	Accelerometer #4	Accelerometer #6
Channel 3	Accelerometer #2	Free	Free
Channel 4	--------------------------Information--------------------------		
Four Span Bridge			
Channel 1	Deflection Gage	Accelerometer #3	Accelerometer #6
Channel 2	Accelerometer #1	Accelerometer #4	Accelerometer #7
Channel 3	Accelerometer #2	Accelerometer #5	Accelerometer #8
Channel 4	--------------------------Information--------------------------		

As a vehicle approached the bridge, the first axle would pass over the first traffic hose which would put a corresponding voltage pulse on Channel 4 on all tapes being recorded. As the first axle passed over the remaining two traffic hoses, corresponding voltage pulses would be recorded on all tapes. Approximately 10 seconds after the vehicle leaves the bridge, the vehicle type code was manually entered and simultaneously recorded on Channel 4 of all tapes. During digitization of the deflection and acceleration records, the first voltage pulse on Channel 4 would be a signal that a vehicle was approaching the bridge, the second pulse would signify that the vehicle was entering the bridge, and the third pulse would be a signal that the first axle was leaving the bridge. The placement of traffic hoses as outlined above automatically precludes the collection of vehicle crossing records in both directions across the bridge. A sample of the output from Channels 1, 2, 3, and 4 for a vehicle crossing a two span bridge can be seen in Figure 2.3. Note the voltage pulses on Channel 4 signifying vehicle position and vehicle position and vehicle type code.

Also see Figure 2.4 for a schematic diagram of the instrumentation for a two span bridge.

Phase I Instrumentation (for data collected during 1973)

Phase I and Phase II instrumentation is identical except for two minor differences. These two differences are the number and location of traffic hoses and the manner in which the vehicle code was recorded.

For Phase I data collection only two traffic hoses were used--one hose at the beginning of the bridge and one at the end. This traffic hose configuration made it possible to record vehicles crossing the bridge in either

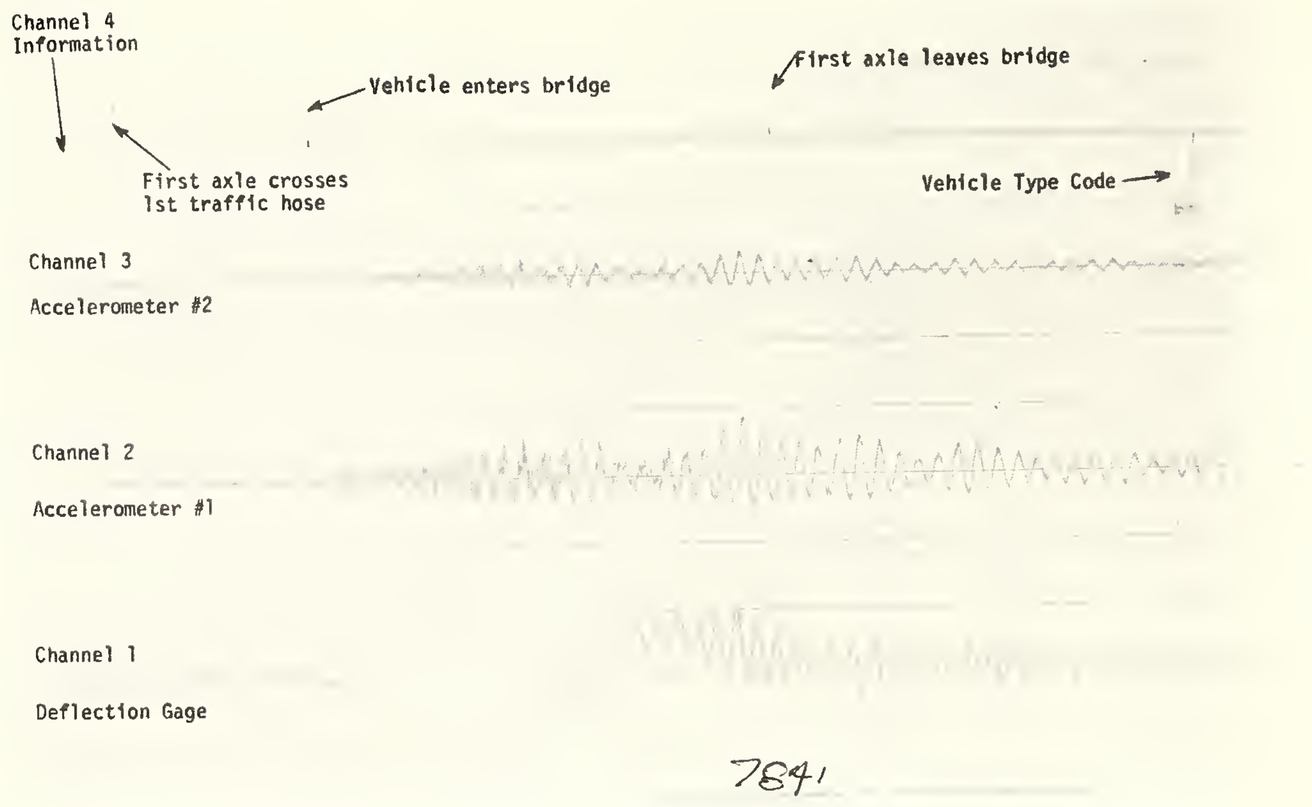


Figure 2.3. Portion of an Oscillographic Strip Chart Showing the Outputs of Channels 1, 2, 3, and 4 for a Vehicle Crossing a Two Span Bridge - Phase II

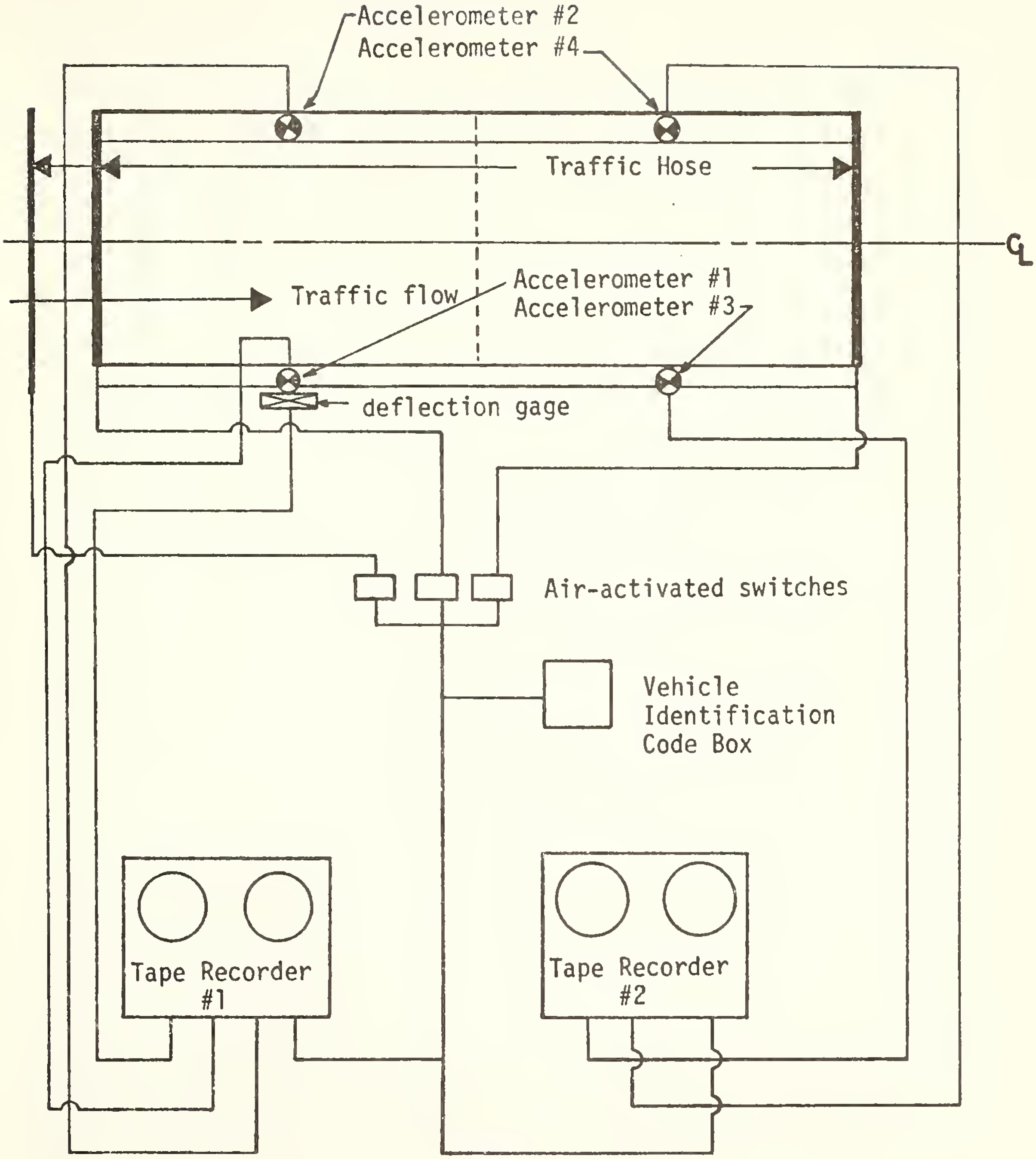


Figure 2.4. Schematic of Phase II instrumentation for a two span bridge.

direction. The approach of a vehicle was recorded by entering the vehicle type code on the tape and after the vehicle left the bridge the vehicle type code was again entered and recorded, thus signifying the end of a vehicle crossing record. A sample vehicle crossing for Phase I can be seen in Figure 2.5. It should be noted that for Phase I all axles crossing the traffic hoses were recorded whereas for Phase II only the first axle crossing was recorded.

The major reason for modifying the instrumentation before Phase II data collection was begun was to decrease the time required to digitize the vehicle crossing records. This will be further explained in Chapter III where the digitization process is described.

2.4.4. Test Procedures

The Indiana State Highway Commission's Research and Training Center conducted the data collection for both Phase I and Phase II. This section will briefly describe the equipment set-up and testing procedures.

The typical bridge testing crew consisted of two engineers and four engineering assistants. The testing of a bridge usually took anywhere from a half day to two days depending upon the type of traffic using the bridge and upon the distance between the bridge site and the Research and Training Center.

Prior to setting up the test equipment, three or four members of the test crew would complete a Human Response Survey Form after standing on the curbs of the bridge under normal traffic conditions. Each person alternately stood at midspan of each span, on both sides of the bridge, remaining at each location long enough to experience the bridge motion under a variety of traffic loading. The human response to bridge motion portion of this

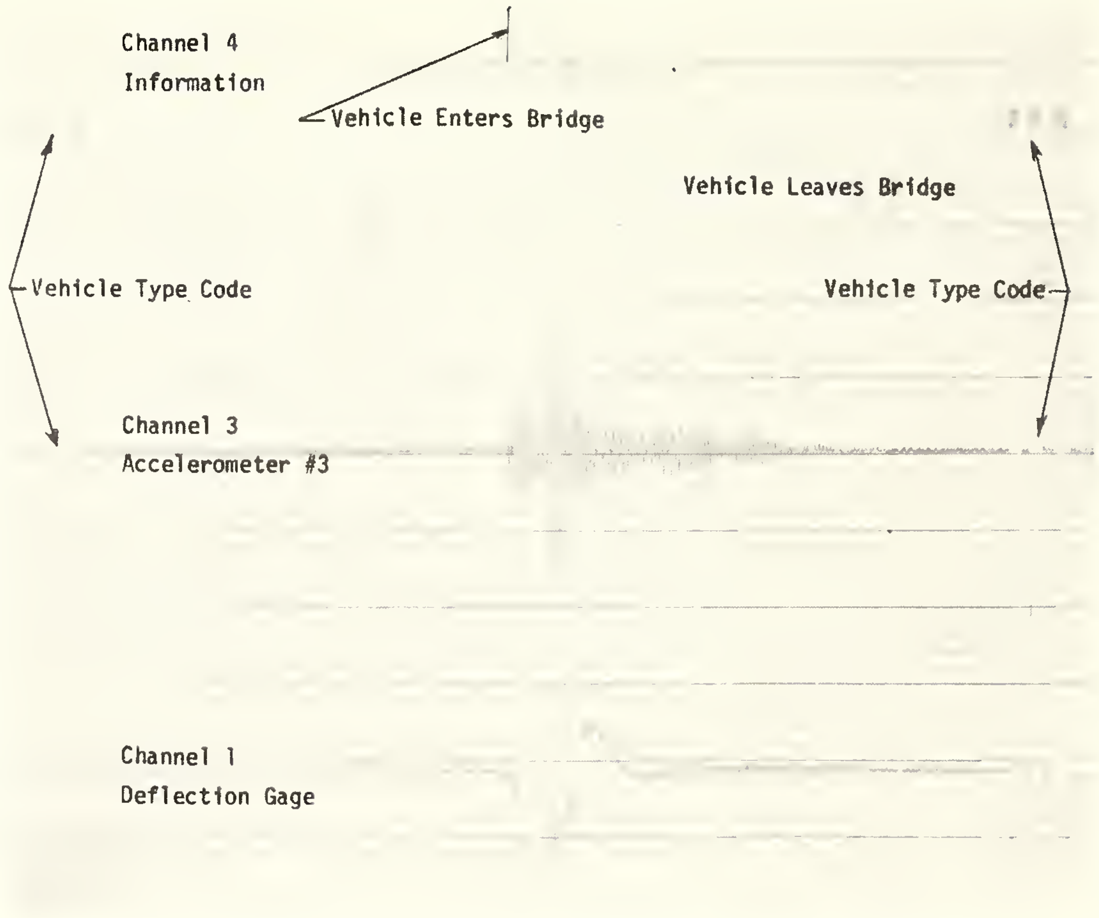


Figure 2.5. Portion of an Oscillographic Strip Chart Showing the Outputs of Channels 1, 3, and 4 for a Vehicle Crossing a Three Span Bridge - Phase I

project is outside the scope of this paper but interested persons may find a complete description of this human response survey, together with results and conclusions in Reference (1).

The testing apparatus was transported to the bridge site in a mobile laboratory (see Figures C.1 and C.2) which was equipped with a portable AC generator. The accelerometers were bolted to preset stainless steel base plates epoxied to the bridge curbs while the deflection gage was placed below the first span and the free end of the cantilevered beam gage was connected to the bridge curb with a tensioned wire. See Figures C.3, C.4 and C.5 in Appendix C for photographs of the accelerometer and deflection gage installations. Appendix C also contains a more detailed explanation of the deflection gage set-up. The traffic hoses were secured to the roadway and all instruments and information switches were connected to the appropriate channels of the tape recorders as described in the previous section.

Before the actual testing was begun, information was gathered concerning the instrumentation location and physical characteristics of the bridge. The general physical characteristics of the bridge included a sketch of the bridge with general physical dimensions. The bridge sketch included an elevation view showing the number of spans and span lengths, a cross section showing the deck width and thickness, beam or girder size and spacing, and a plan view showing the bridge orientation, skew, and location of test instuments. Also a general inspection of the structural condition of the bridge was conducted and recorded. The clearance between the deck and approach slab was measured and recorded along with the condition of the joint and joint material. The top and bottom of the deck along with the beams or girders were

inspected for deterioration or corrosion. If any abnormalities were present, the general extent and location were recorded. Also, the bearing supports were inspected for freedom of movement since rusting of bearing plates or rockers could result in a frozen bearing support which could affect the dynamic motion of the bridge. In general any condition which would affect the motion of the bridge structure was documented at this time.

The actual testing was begun after the tape recorders were turned on and a verbal description of the bridge, its location, weather conditions and the instrumentation information were recorded on Channel 4. Next, the deflection gage and each accelerometer were calibrated with the calibration signals recorded on the appropriate tape recorder channel. The testing could now begin.

During the test of a bridge all test personnel had to be off the bridge and approaches and, if at all possible, they were in a position so they could not be seen by passing vehicle drivers. Only one man was stationed near the bridge so as to be in position to observe the approaching traffic. However, this "vehicle spotter" was in constant voice communication with the test personnel in the mobile lab who were operating the tape recorders and the vehicle type code box.

The following will be a brief description of the procedure followed for collecting a single vehicle crossing for Phase I and Phase II. Note that in most cases the tape recorders were operated continuously.

Phase I Test Procedure

As a vehicle approached the bridge under favorable conditions, which will be discussed later, the vehicle spotter informed the crew in the mobile lab who entered the corresponding vehicle code, (see Table 2.2) preferably

at least 5 seconds before the vehicle entered the bridge. Simultaneously, a member of the test crew in the mobile lab would record on a log sheet a description of the vehicle, the lane (right or left) if the bridge was one way, the direction of travel if a two way bridge, and the tape footage before and after the vehicle crossed the bridge. After the vehicle left the bridge the operator in the mobile lab would wait for approximately 10 seconds before entering the vehicle code which signified the end of a record.

Phase II Test Procedure

The test procedure for Phase II is essentially the same as that for Phase I except for the following changes. If the record for a vehicle approaching the bridge during Phase I testing was not to be collected, the traffic hoses remained activated so tire voltage spikes would appear on Channel 4, although there would be no vehicle codes entered before or after the vehicle crossed the bridge. In Phase II testing the traffic hoses were only activated when a vehicle crossing was desired. If the record for a vehicle approaching the bridge was desired, the traffic hoses were activated so the first tire spike on Channel 4 signified the beginning of the record. After the vehicle left the bridge the vehicle code box had a built-in time delay device which would prohibit the operator from entering the vehicle code for at least 10 seconds. No log sheets were kept because the vehicle code types had been expanded from 4 for Phase I to 9 for Phase II and the direction of travel was dictated by the traffic hose configuration. For Phase II data, the lane of travel is unknown, although examination of the deflection record or magnitudes of acceleration usually indicates in which lane the vehicle was traveling.

Required Conditions for a Vehicle Crossing to be Collected

At the beginning of the discussion on the Phase I test procedures, it was mentioned that the vehicle spotter would decide if the vehicle was approaching the bridge under favorable conditions. The following will be a discussion of these "favorable conditions" along with other criteria affecting the decision as to whether a vehicle crossing would be recorded. In order to be able to compare the test results with the results from the analytical programs, the bridge had to be as stationary as possible before the vehicle entered the bridge. For the same reason only one vehicle at a time was allowed on the bridge. Thus, the vehicle spotter had to decide if the bridge would be relatively quiescent when a vehicle entered the bridge and also judge whether there was sufficient time for the bridge to vibrate freely for at least 10 seconds before another vehicle would enter the bridge. The 10 second delay after the vehicle left the bridge was required in order to obtain a sufficient number of points for the Fourier decomposition of the free vibration portion of either the deflection or acceleration records.

Also, the test crew had to collect a representative sample of vehicle types crossing a bridge. For a county road this sample would consist mainly of passenger cars and farm trucks. For an Interstate or heavily traveled U.S. highway the test crew, in addition to collecting a broad range of vehicle types, were required to have the total vehicle crossing sample for a particular bridge consist of at least 60% truck crossings (vehicle Code 4 for Phase I, Vehicle Codes 4 through 9 for Phase II). This was done to satisfy one of the objectives of this study which was to ascertain what range of maximum response could be expected from a particular bridge structure.

Test Vehicle (Research and Training Center Bus)

As was discussed in Section 2.3.1, a test vehicle was used in order to provide more confidence when comparing analytical results with test results and also to provide a vehicle of sufficient weight to excite bridges that had light, low weight traffic. This test vehicle was the Research and Training Center bus loaded with broken concrete beams.

Testing each bridge with the test vehicle was integrated with the normal data collection during both Phases of testing. No problems were encountered if the traffic volume was light. However, when the traffic volume was heavy, vehicles tended to bunch up around the R & TC bus, thus causing difficulties in collecting a record where the bridge was stationary and where no vehicles entered the bridge for 10 seconds after the bus left the structure.

The type of roadway and physical characteristics of the roadway, bridge and approach determined the range of bus speeds used during the testing. Since all 62 bridges in the program were tested using the R & TC bus, certain standard speeds for the different categories of bridges were established. For example, on county roads crossing over Interstate highways, bus speeds of 10, 20, 30 and 40 mph were used because there was no danger of impeding existing traffic at the slower test speeds. On some county road bridges it was possible to conduct 50 mph test runs, but on others, horizontal and vertical curves directly before the bridge approach created safety hazards for vehicles traveling at 50 mph. On state highways, test speeds of 40 and 50 mph were used. This included at least two test runs at each speed, one in each direction on highways with two-directional traffic (Phase I only), and one in each lane on four-lane divided highways. On

Interstate bridges, a minimum of 3 test runs at 50 mph in the right lane were conducted.

2.4.5. Special Tests

Special tests were conducted after the conclusion of Phase II data collection using the test vehicle to excite the bridge. These special tests were conducted because various questions had arisen after a portion of the data had been reduced. These questions concerned results from the analytical programs and the need to check some portions of the computer program used in the data reduction. A more detailed explanation of these special tests may be found in the next chapter while the detailed testing procedure may be found in Appendix D.

For the special tests, three bridges were chosen from among the 62 bridges selected for testing. These three bridges had to be easily accessible for testing and their section properties compatible with the capabilities of the analytical programs. Those bridges selected were the single span SB-C-1, the two span KCSG-A-1 and the three span CSB-C-1, all of which were steel beam or plate girder bridges with constant moment of inertia.

The special tests were divided into three parts: Test A; Test B; and Test C. Test A was conducted to check the effects of vehicle velocity on results of the double integration of the acceleration record while Test B checked the variability of test results. Test C was used to experimentally determine the torsional and certain bending frequencies excited by vehicles crossing a bridge structure. See Appendix D for more details.

CHAPTER III: REDUCTION OF THE TEST DATA

3.1. General

More than 2200 individual vehicle crossings were collected during Phase I and Phase II testing, constituting over 13,000 deflection and acceleration records which had to be reduced. Due to the cost of computer time used to reduce the data, only 900 vehicle crossings were actually digitized and stored on magnetic tape. Of these 900 vehicle crossings, approximately 65% were truck crossings, 30% test vehicle crossings, and 5% passenger car and light vehicle crossings.

In performing any data analysis digitally, it is first necessary to obtain discrete samples of the data. The phenomenon being observed in the test program is continuous and these observations are recorded in a continuous manner on magnetic tape. The continuous data must then be sampled to provide the discrete values required for digital analysis. This sampling process is called digitization.

This chapter will describe the process of reducing the data, from digitization through the computer programs which were used to find the deflection, velocity, acceleration, jerk and frequency content for the vehicle crossings.

3.1.1. Data Classification

Any observed data may be categorized broadly as either deterministic or nondeterministic in nature. Deterministic data are those which can be described accurately by an

explicit mathematical relationship. An example of this type of data would be the motion of a point located on the perimeter of a wheel rotating at a constant angular frequency. Nondeterministic or random data are those observations which cannot be described by an explicit mathematical relationship where the only meaningful statements which can be made about such data are statistical or probabilistic in nature. An example of a random phenomenon is the acoustic noise of a jet aircraft engine.

The classification of data into random and deterministic categories is open to question in that observations taken of physical phenomena are almost never truly one or the other. Instead, deterministic data usually contain a random component due to measurement errors, environmental effects, etc., or what may appear to be random data may actually be deterministic data whose mathematical relationship are so complex as to be unknown at the present time.

The data collected for this project will be considered as deterministic and transient (decaying with time) even though the present state of the art cannot exactly predict the motion of a bridge excited by a vehicle crossing.

3.2. Digitization of Data

In order to take advantage of the speed and versatility of the Dual CDC 6500 digital computer facilities available at Purdue, the data, which were stored in analog (continuous) form, had to be converted to digital form. This is accomplished by an analog-to-digital converter (A/D converter), which takes the voltage time histories from the ¼" magnetic tapes and approximates this continuous signal by a specified number of discrete points per time period. The A/D conversion was performed in the Applied Stochastics Laboratory operated by the School of Mechanical Engineering under the direction of Professor Anshel Schiff.

The equipment used to digitize and store the vehicle crossing data consisted of one of the Tandberg tape recorders used to record the data, a 100 Hz low pass filter, a data acquisition system (DAS) which includes the A/D converter, a mini-computer system which includes a 9-track tape drive, a disk system, a line printer, and CRT terminal with a keyboard and display. A block diagram of the equipment set-up may be seen in Figure 3.1. For a more detailed description of the preceding equipment and digitization procedures see Appendix E.

3.2.1. Digitization Procedures

The following section will briefly describe how a typical vehicle crossing record was digitized and stored on magnetic tape. The procedure described will be for both Phase I and Phase II data. For the sake of clarity, a vehicle crossing record (or just record) will refer to all data collected from all transducers for a particular vehicle crossing the bridge. A file will refer to the data collected from only one transducer for a vehicle crossing a bridge.

The equipment was set up as shown in Figure 3.1 and the original field tape was mounted on the tape recorder. The digitization time or sampling interval, which was 5 milli-seconds (200 points/second), was dialed into the DAS. The rationale behind this choice of sampling interval may be found in the appropriate section of Appendix E. The calibration files were digitized separately, followed by each vehicle crossing record. When a vehicle crossing record was found on the field tape, the following information was entered at the CRT terminal: (1) the field tape number; (2) the estimated length in seconds of the vehicle crossing; (3) bridge ID and description; and (4) vehicle description (Phase I only). The tape recorder was then

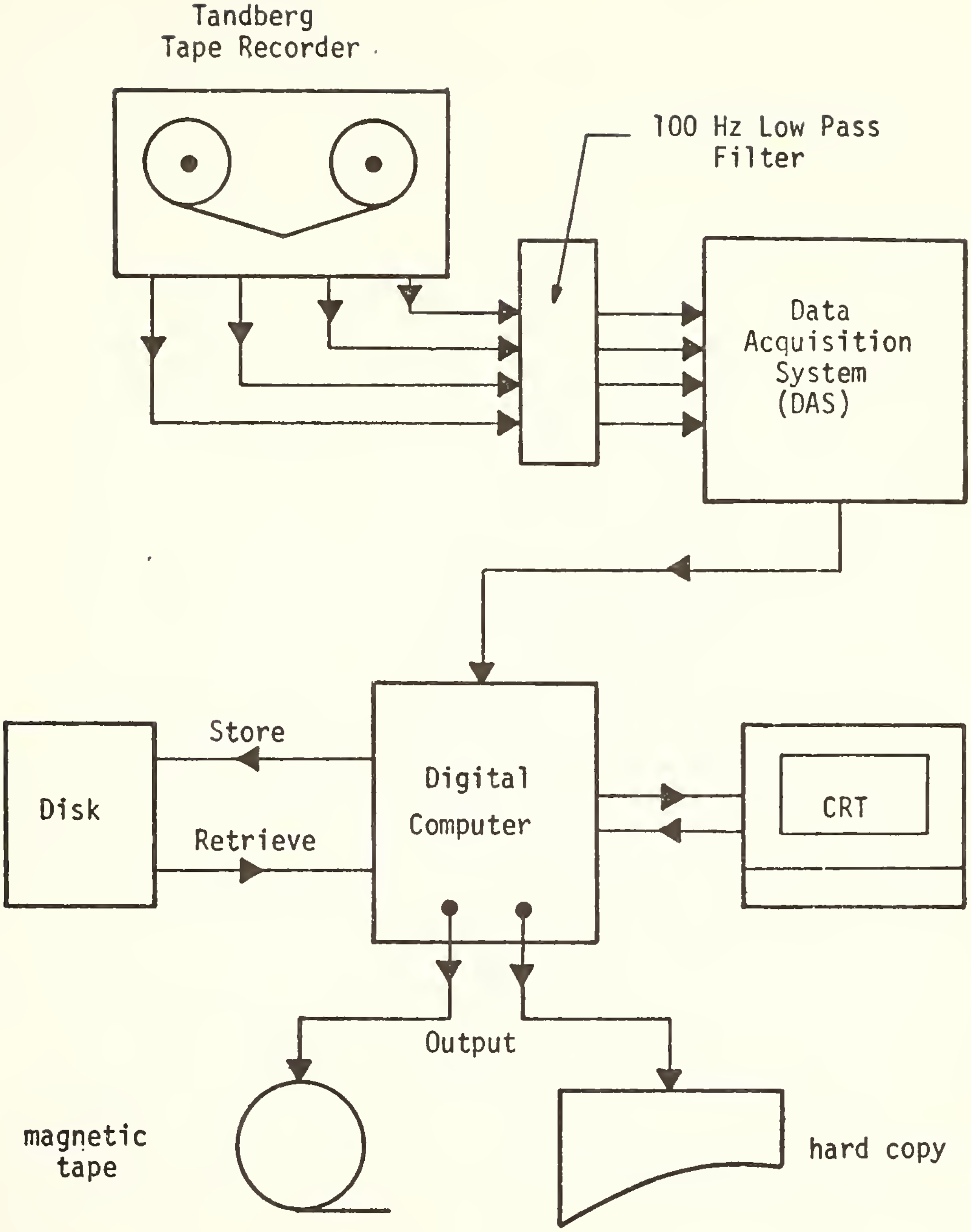


Figure 3.1. Equipment set-up for digitizing the data.

turned on, the DAS was activated, and the output from all four channels of the tape recorder was simultaneously digitized and stored on disk. The tape recorder and DAS were turned off at the end of the vehicle crossing and the computer processed all four channels and stored the three data channels on 9-track magnetic tape. The computer also assigned a unique consecutive file number to each file as it was stored on tape and also printed out information concerning the files just digitized. If the data digitized were from the field tape recorded on tape recorder #1 (See Table 2.4), the 9-track tape would contain three files, one deflection file followed by the files for Accelerometers 1 and 2.

Each data file stored on the magnetic tape was preceded by a header which contained information concerning that particular file. In addition to the quantities entered at the CRT Terminal, the header contained the number of data points contained in the file and the number of points into the file where the first axle enters the bridge, where the last axle enters the bridge (Phase I only), where the first axle leaves the bridge, and where the last axle leaves the bridge (Phase I only). More information concerning the header and data format may be found in Section E.2.2 of Appendix E.

The 9-track tape containing the digitized data files was written in IBM's 9-track EBCDIC. Since there is no 9-track capability on Purdue's CDC 6500 system, the 9-track was converted to IBM's 7-track BCDIC which is compatible with the Purdue system. Inasmuch as the 9-track tapes were not destroyed, the actual field data is stored in three forms, the original ¼" x 1800 feet field tapes, IBM's 9-track EBCDIC, and IBM's 7-track BCDIC. Thus, the data are readily available to other researchers or interested persons who might want to conduct further investigation into these vehicle crossing records.

3.3. Analysis of the Test Data

The main objective of this investigation is to reduce and analyze the data collected, in order to establish basic response characteristics for the different types of bridge structures. The response characteristics under consideration are the maximum deflection, velocity, acceleration, jerk and frequency content. The importance in obtaining the range of velocity, acceleration and jerk values for various bridge types is their relationship to the comfort of pedestrians or persons in halted vehicles on a bridge which is moving. Shahabadi(1), in his human comfort to bridge motion studies, expresses the fact that velocity, acceleration and jerk adversely effect human comfort at different frequency ranges. Thus, it is advantageous to find the maximum deflection, velocity, acceleration, jerk and frequency content for each vehicle crossing record in order to identify which response characteristics(s) is (are) controlling the motion of a bridge with respect to human comfort.

In order to obtain the maximum velocity or jerk, algorithms had to be developed which would either differentiate the deflection or acceleration file or integrate the acceleration file since these quantities were known. To attain this goal, an accelerometer was always positioned adjacent to the deflection gage during the testing of a bridge. In this manner integration or differentiation techniques could be developed and directly checked with the actual values. For example, suppose the deflection file for a certain vehicle crossing is smoothed using a particular polynomial and number of points and subsequently differentiated twice. By varying the degree of polynomial and number of points and calculating the variance between the actual acceleration and the twice differentiated deflection ,the most efficient smoothing

polynomial might be chosen. In the same manner the best integration techniques may be selected.

Several problems arise if a raw data file is differentiated or integrated without regard to the errors introduced during these processes. If a data file is differentiated the random noise inherent in the digitized signal is magnified, which makes smoothing of the data before differentiation a necessity. The degree of smoothing is critical since too much will cause the loss of valid data and too little will introduce spurious results into the differentiated data. On the other hand, if a raw data file is integrated the effect of the random portion of the data will be reduced. However, the problem which must be dealt with is the fact that the initial conditions are not zero and are unknown. The methods developed for differentiation and integration are developed and presented in Appendix F.

Figures 3.2 through 3.9 present the results of the integration techniques which were developed. It should be noted that although the twice integrated acceleration does not exactly follow the actual displacement, the maximum displacement from acceleration which is of primary interest is always within an acceptable range. A different algorithm had to be developed depending on the number of spans the bridge contained; Figures 3.2 and 3.3 show the results for a single span bridge, Figures 3.4 and 3.5 for a two span bridge, Figures 3.6 and 3.7 for a three span bridge, and Figures 3.8 and 3.9 for a four span bridge.

Therefore, an accelerometer could be attached to a bridge deck and the records of vehicle crossings recorded and digitized. Using the appropriate integration techniques developed and presented in Section F.4 of Appendix F, the displacement or velocity curve for each vehicle crossing could be obtained, though it would be recommended that the

results of at least five crossing records for each type of vehicle be collected before any conclusions concerning the maximum displacements are reached. This should be done since the techniques are very sensitive to the bridge being in a quiescent state as a vehicle enters the bridge.

Figures 3.10 through 3.13 present typical results for differentiating the displacement to obtain the acceleration for a one through four span bridge respectively. The smoothing polynomial applied to the displacement data which gave the best results for all bridge and vehicle types was a third degree polynomial using nine points, four points on either side of the point where the derivative is being calculated. If, for some reason, the data contains a more than normal amount of noise, a third degree polynomial using between thirteen and thirty points would give satisfactory results. More specific results will be presented in Chapter 4.

3.3.1. Computer Programs Used in Data Analysis

Two computer programs were written to facilitate the reduction and analysis of the test data. Both programs were essentially the same except for the input format changes which were necessary due to the difference in the instrumentation in Phase I and Phase II data collection. The programs were designed to read the data files directly from the magnetic tape, process each file, and plot the results. The following is a list of the functions performed by the programs:

(1) calculate the calibration constants,

(2) find the log decrement and damping ratio of the bridge using both deflection and acceleration files,

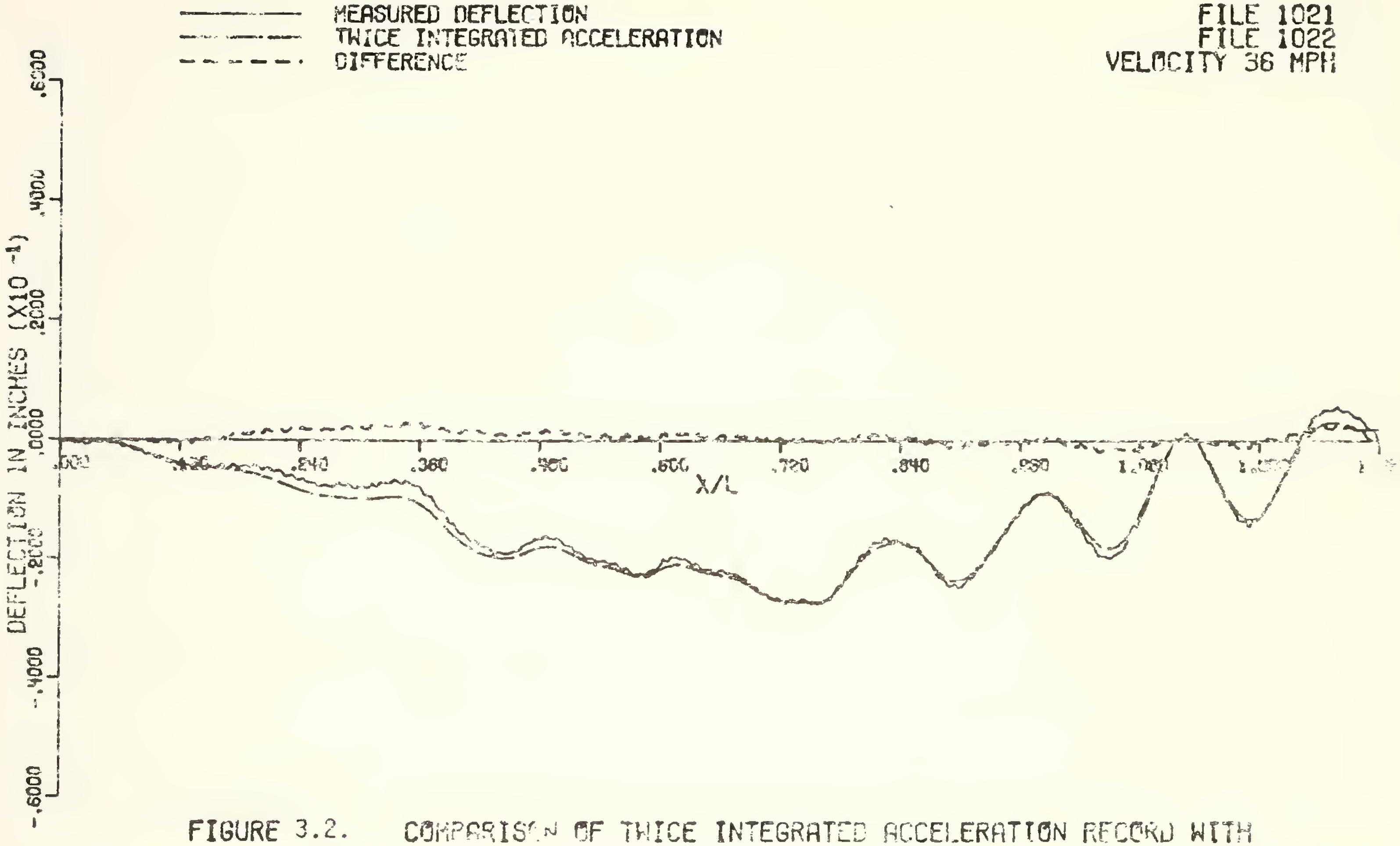


FIGURE 3.2. COMPARISON OF THICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

SINGLE SPAN STEEL BEAM BRIDGE
SPAN LENGTH 72.0FT
WIDTH - 41.0FT
US 231 OVER HOWE DITCH JASPER COUNTY
BRIDGE STUDY NUMBER SB-C-1

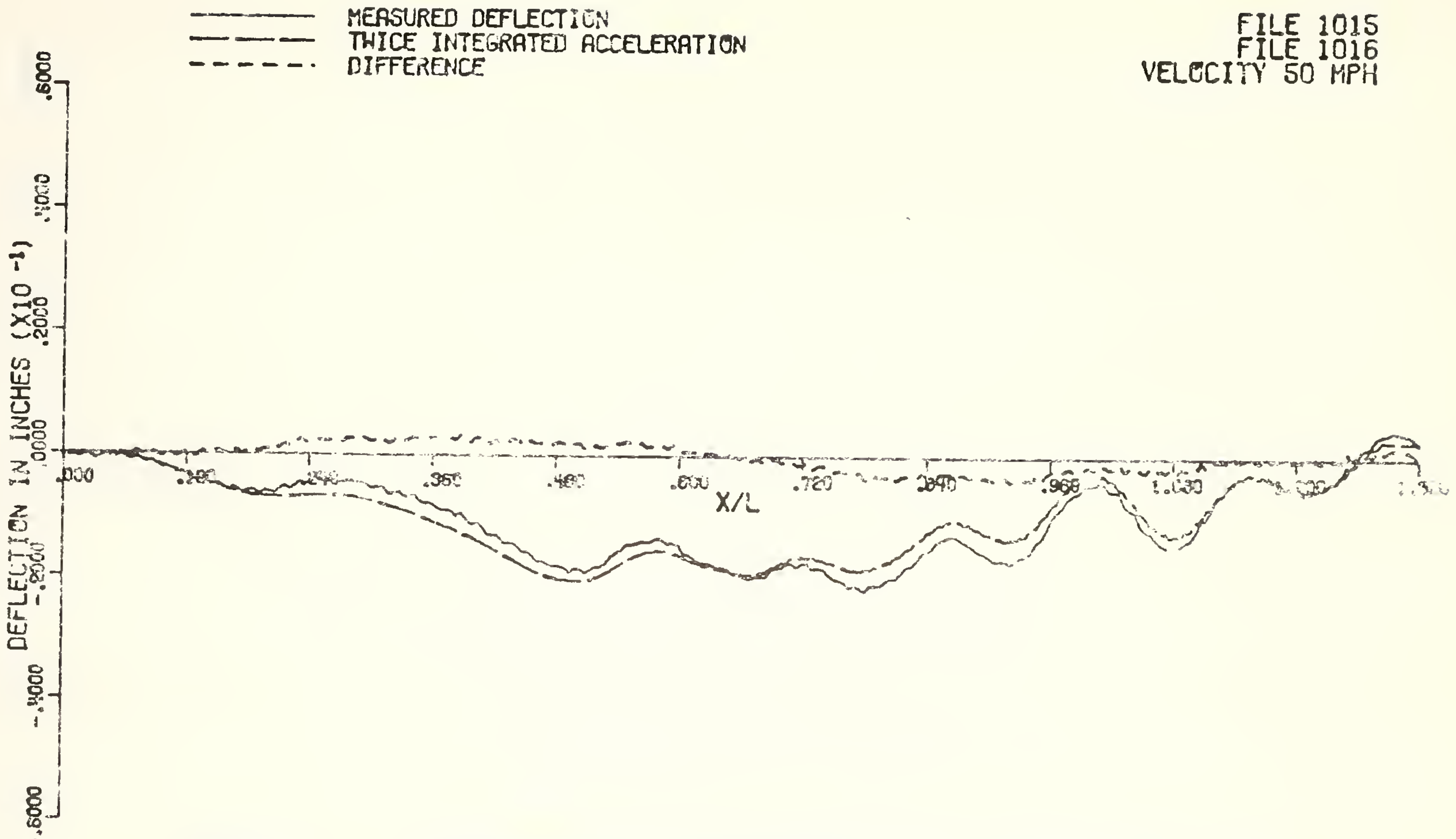


FIGURE 3.3. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

SINGLE SPAN STEEL BEAM BRIDGE
SPAN LENGTH 72.0FT
WIDTH - 41.0FT
US 231 OVER HOWE DITCH JASPER COUNTY
BRIDGE STUDY NUMBER SB-C-1

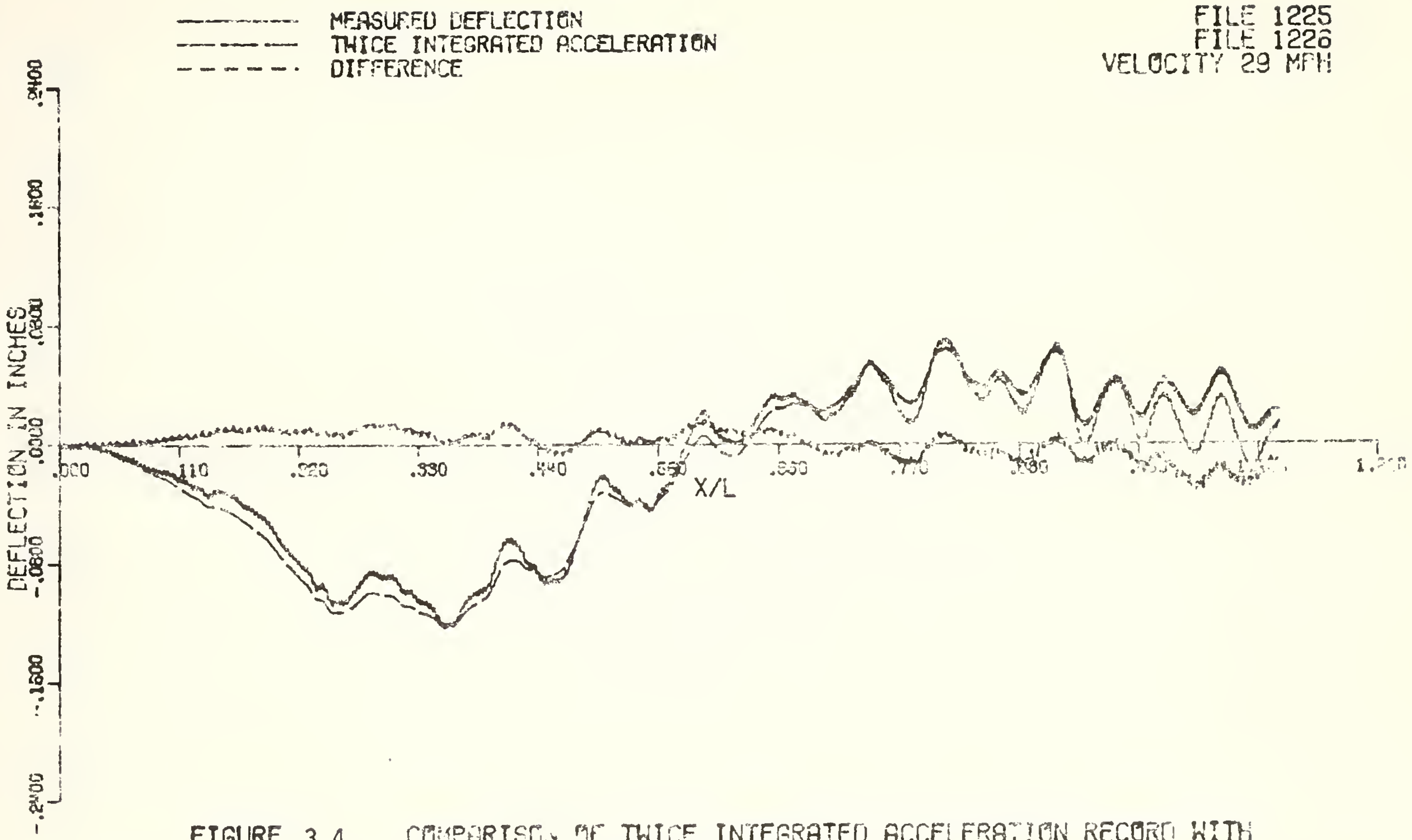


FIGURE 3.4. COMPARISON OF THICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

2-SPAN COMPOSITE CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 96.0FT,96.0FT
WIDTH - 25.5FT
COUNTY ROAD 1200S OVER I-65 JASPER COUNTY
BRIDGE STUDY NUMBER KCSB-D-1

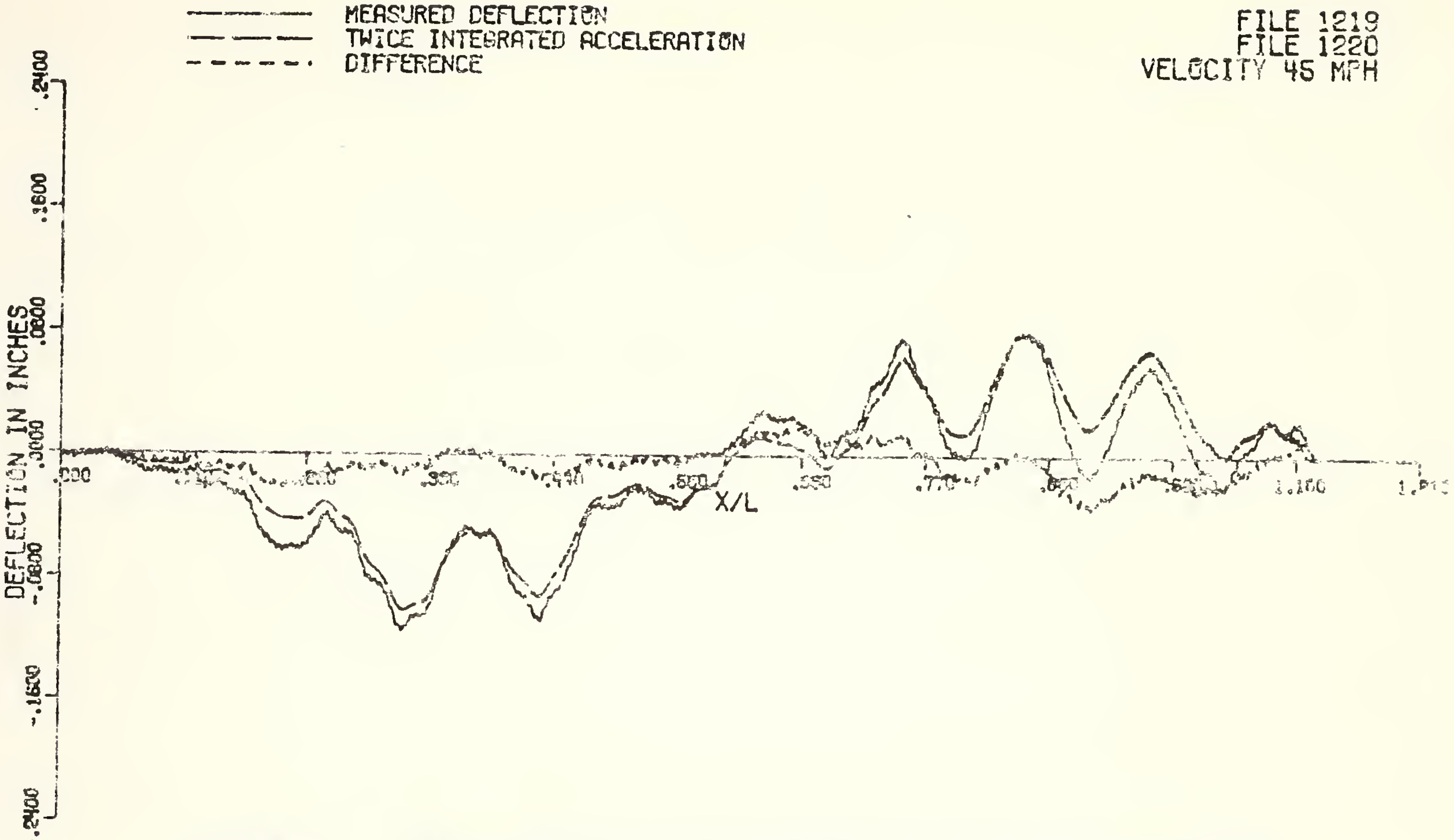


FIGURE 3.5. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

2-SPAN COMPOSITE CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 96.0FT,96.0FT
WIDTH - 25.5FT
COUNTY ROAD 1200S OVER I-65 JASPER COUNTY
BRIDGE STUDY NUMBER KCSB-D-1

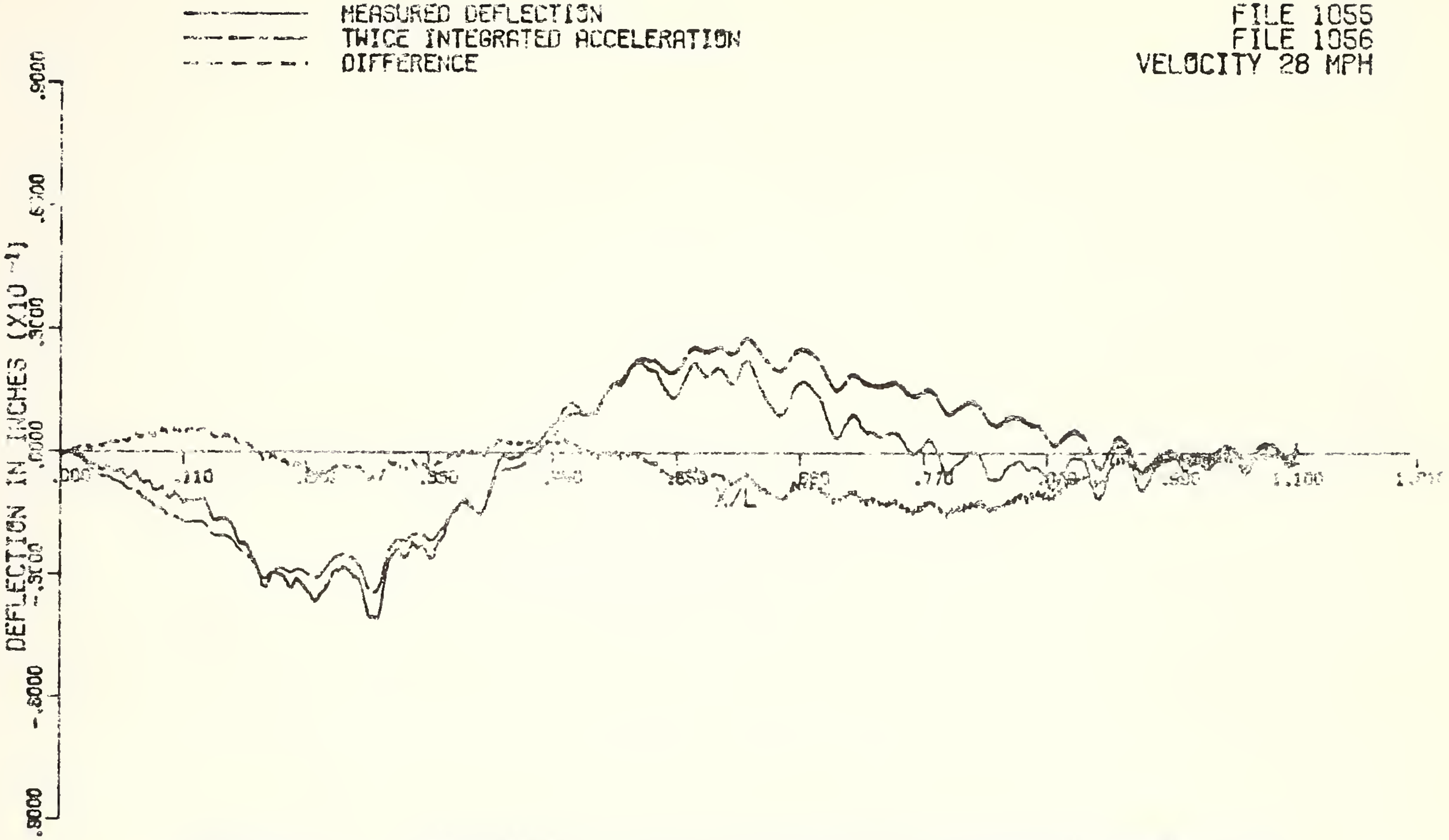


FIGURE 3.6. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

3-SPAN CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 68.0FT,85.0FT,68.0FT
WIDTH - 31.5FT
US 41(SB) OVER IROQUOIS RIVER NEWTON COUNTY
BRIDGE STUDY NUMBER CSB-C-1

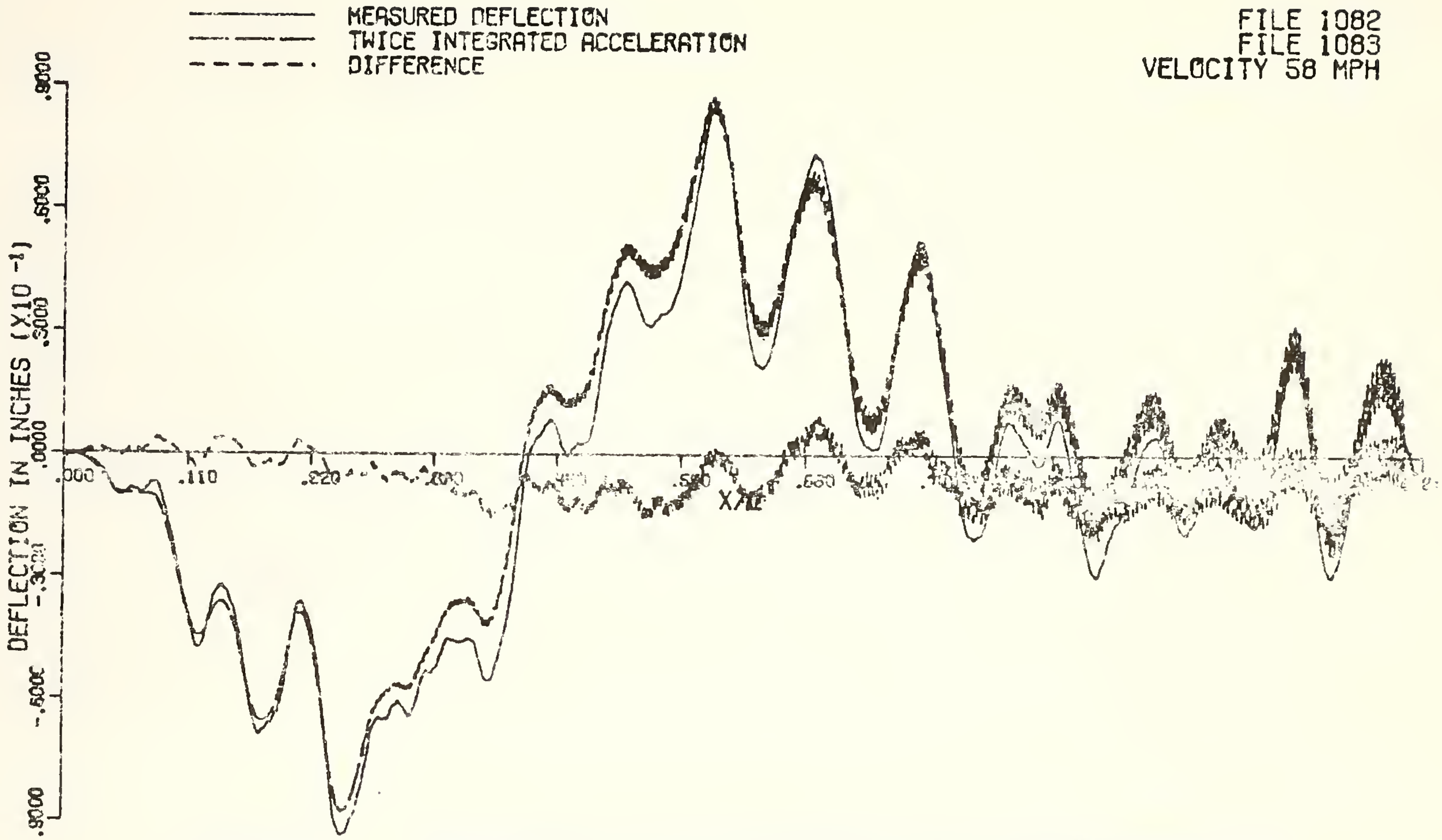


FIGURE 3.7. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

3-SPAN CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 68.0FT.85.0FT.68.0FT
WIDTH - 31.5FT
US 41(SB) OVER IROQUOIS RIVER NEWTON COUNTY
BRIDGE STUDY NUMBER CSB-C-1

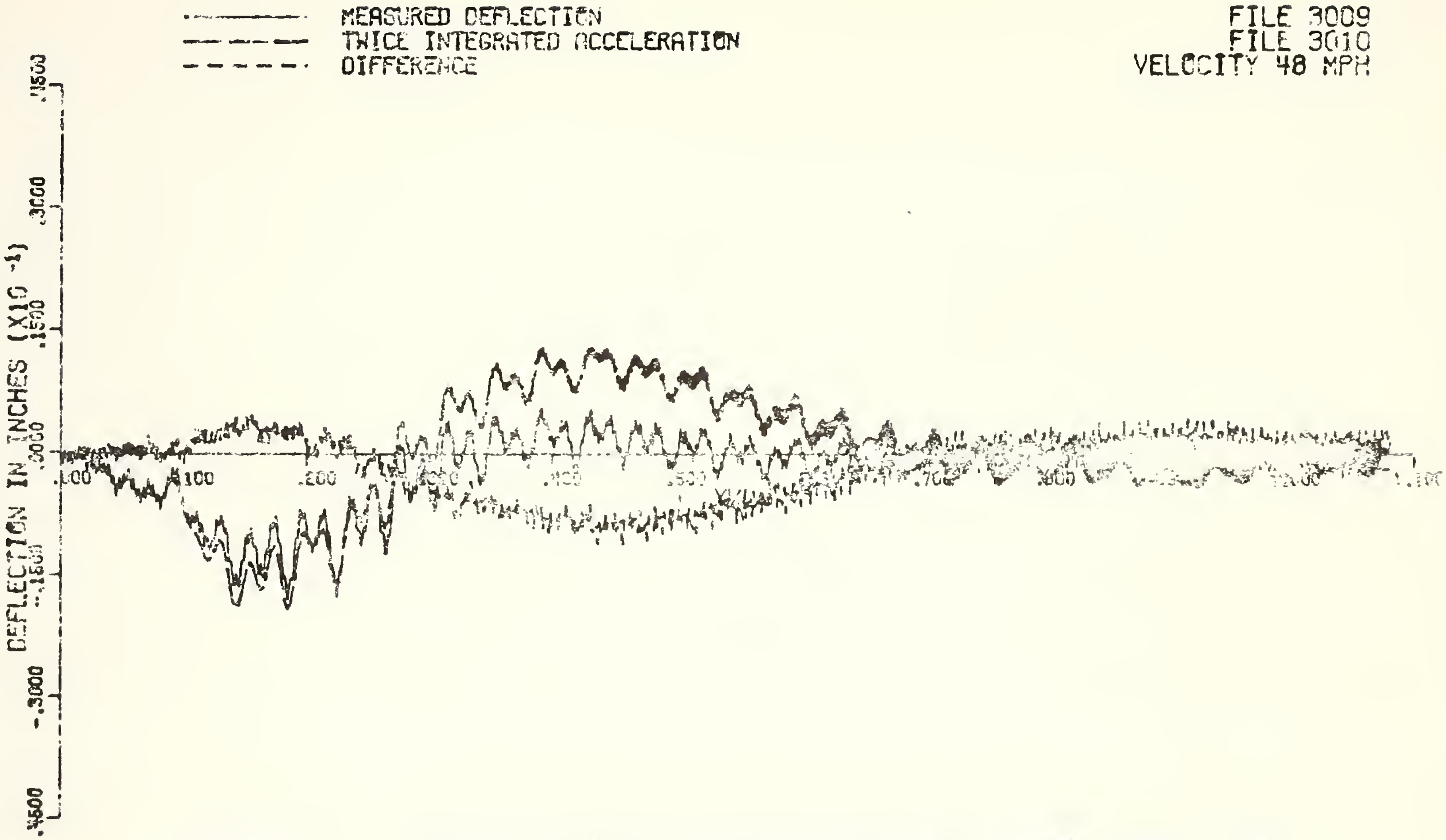


FIGURE 3.3. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

4-SPAN PRESTRESSED CONCRETE I-BEAM BRIDGE
SPAN LENGTHS 70.75FT.72.0FT.72.0FT.70.75FT
WIDTH - 30.0FT
US 31(NB) OVER EEL RIVER MIAMI COUNTY
BRIDGE STUDY NUMBER PCIB-A-2

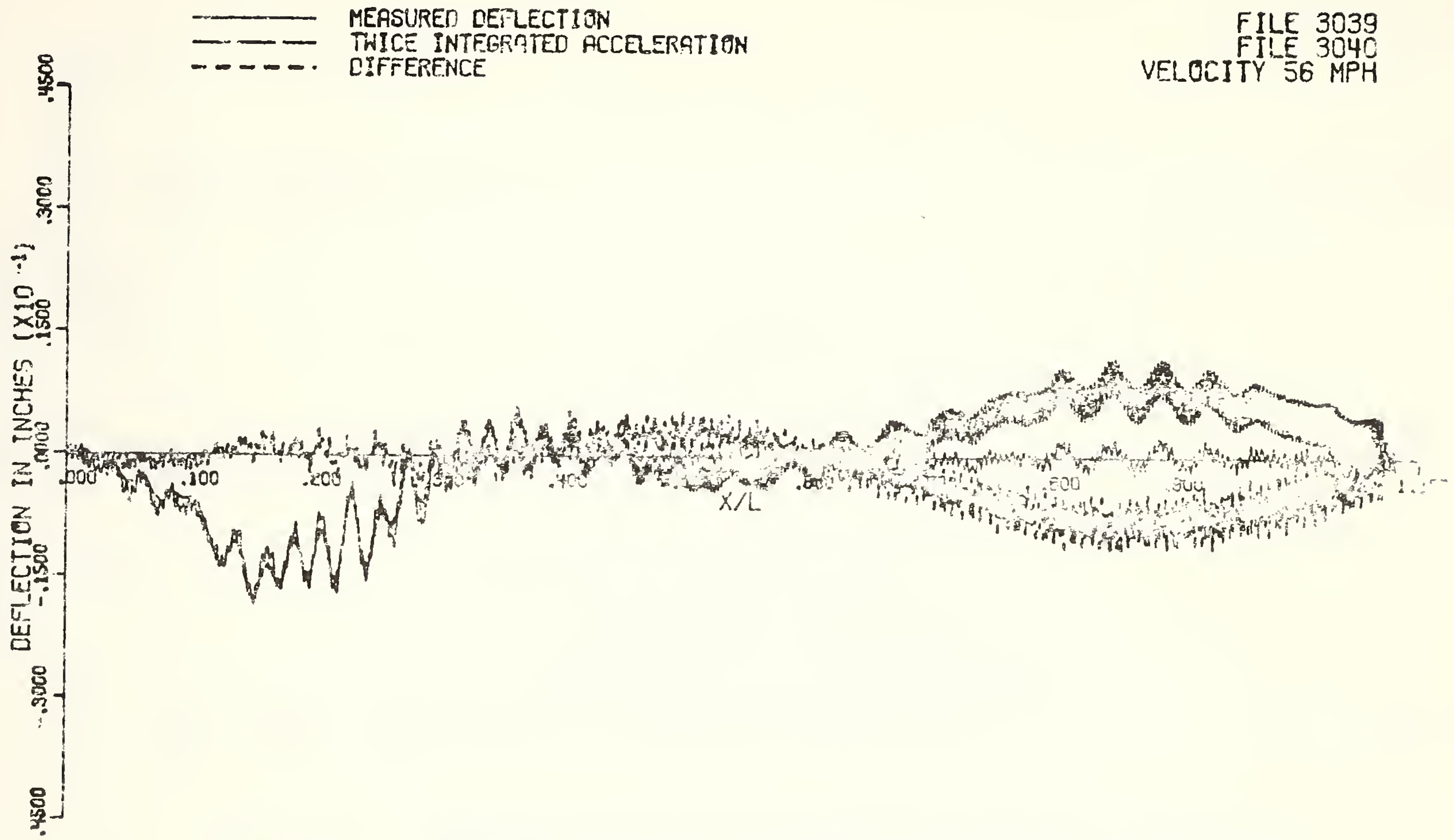


FIGURE 3.9. COMPARISON OF TWICE INTEGRATED ACCELERATION RECORD WITH THE ACTUAL DEFLECTION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

4-SPAN PRESTRESSED CONCRETE I-BEAM BRIDGE
SPAN LENGTHS 70.75FT.72.0FT.72.0FT.70.75FT
WIDTH - 30.0FT
US 31(NB) OVER EEL RIVER MIAMI COUNTY
BRIDGE STUDY NUMBER PCIB-A-2

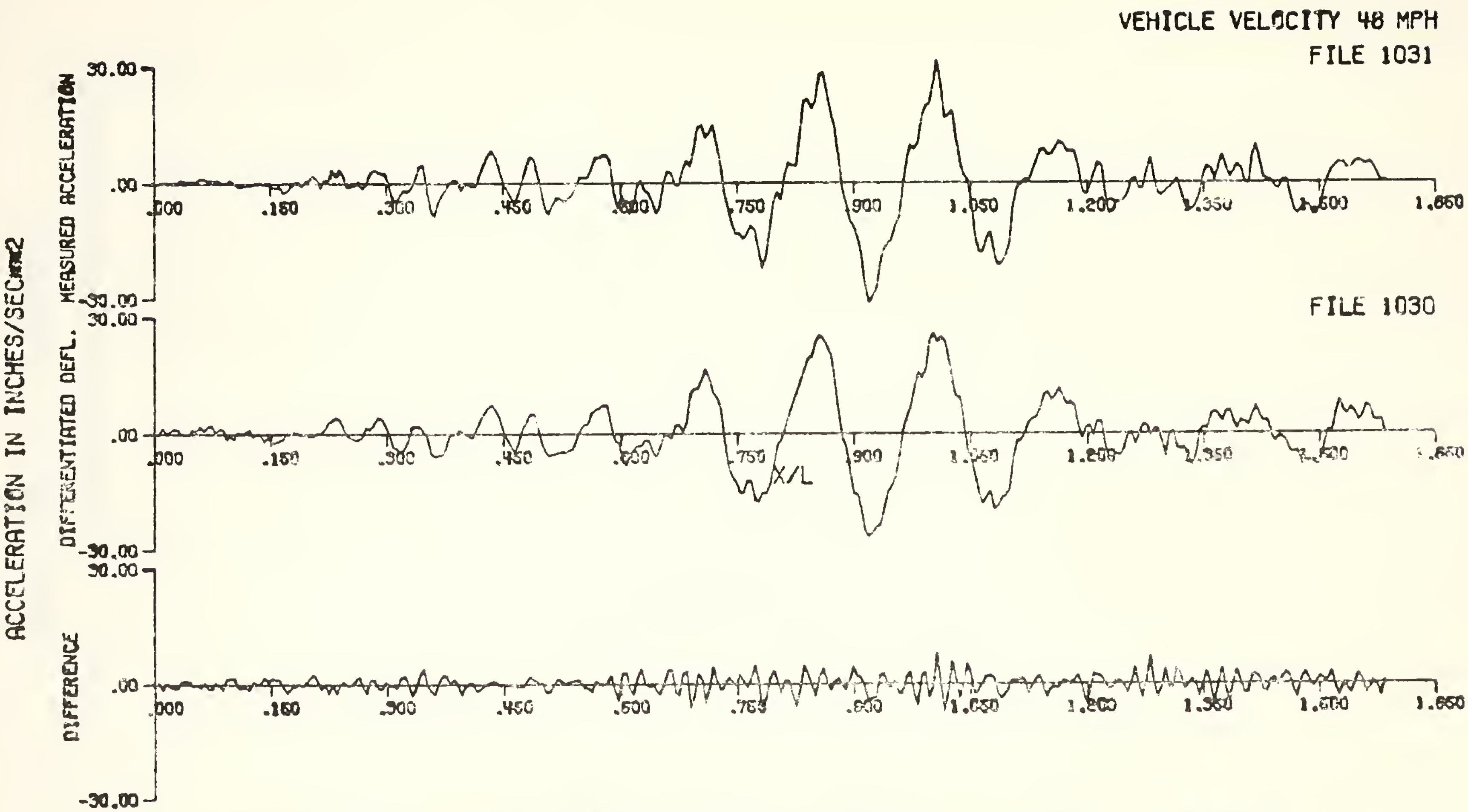


FIGURE 3.10. COMPARISON OF TWICE DIFFERENTIATED DEFLECTION RECORD WITH THE ACTUAL ACCELERATION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

SINGLE SPAN STEEL BEAM BRIDGE
SPAN LENGTH 72.0FT
WIDTH - 41.0FT
US 231 OVER HOWE DITCH JASPER COUNTY
BRIDGE STUDY NUMBER SB-C-1

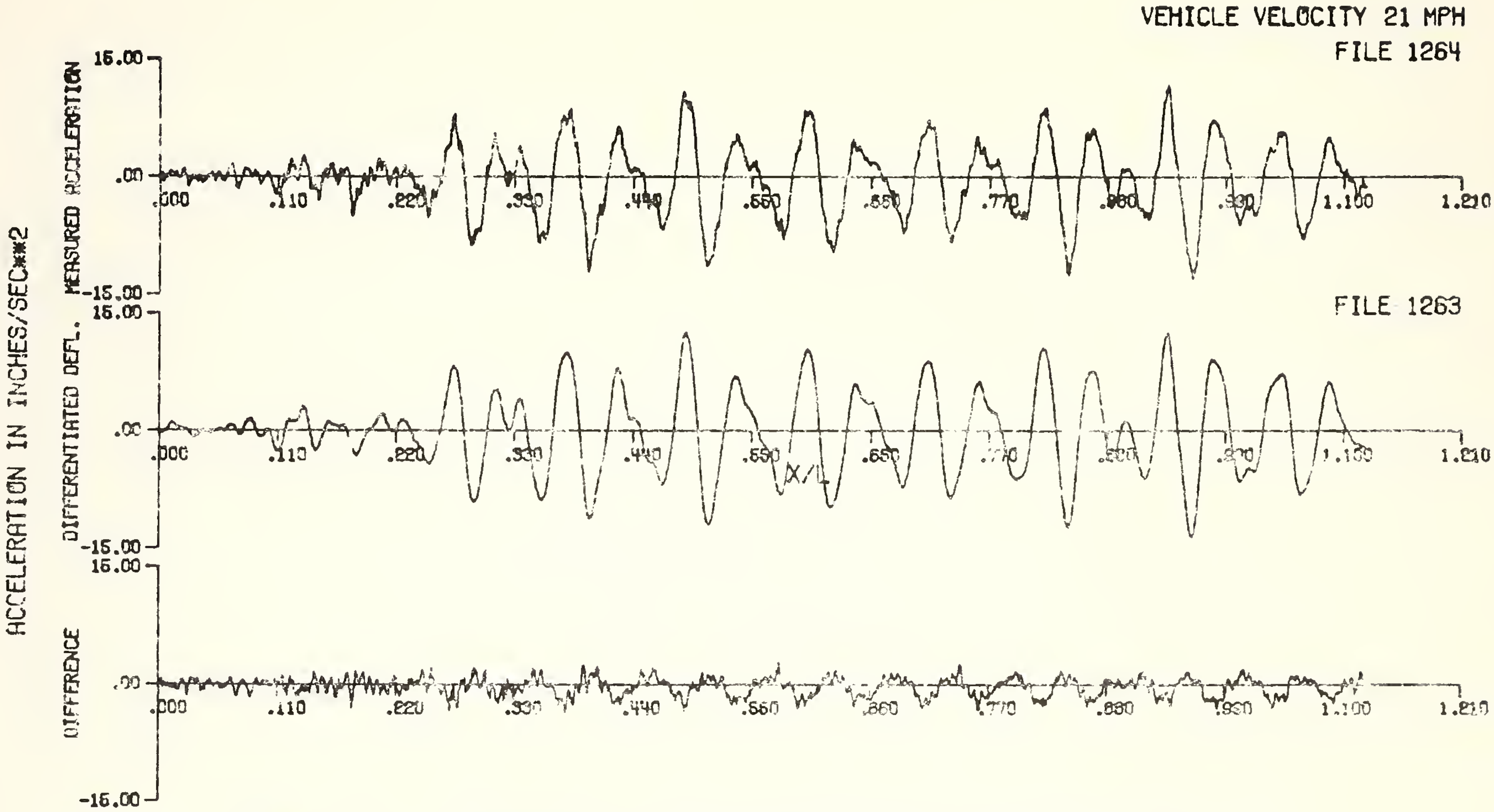


FIGURE 3.11. COMPARISON OF TWICE DIFFERENTIATED DEFLECTION RECORD WITH THE ACTUAL ACCELERATION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

2-SPAN COMPOSITE CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 96.0FT.96.0FT
WIDTH - 25.5FT
COUNTY ROAD 400N OVER I-65 JASPER COUNTY
BRIDGE STUDY NUMBER KCSB-D-2

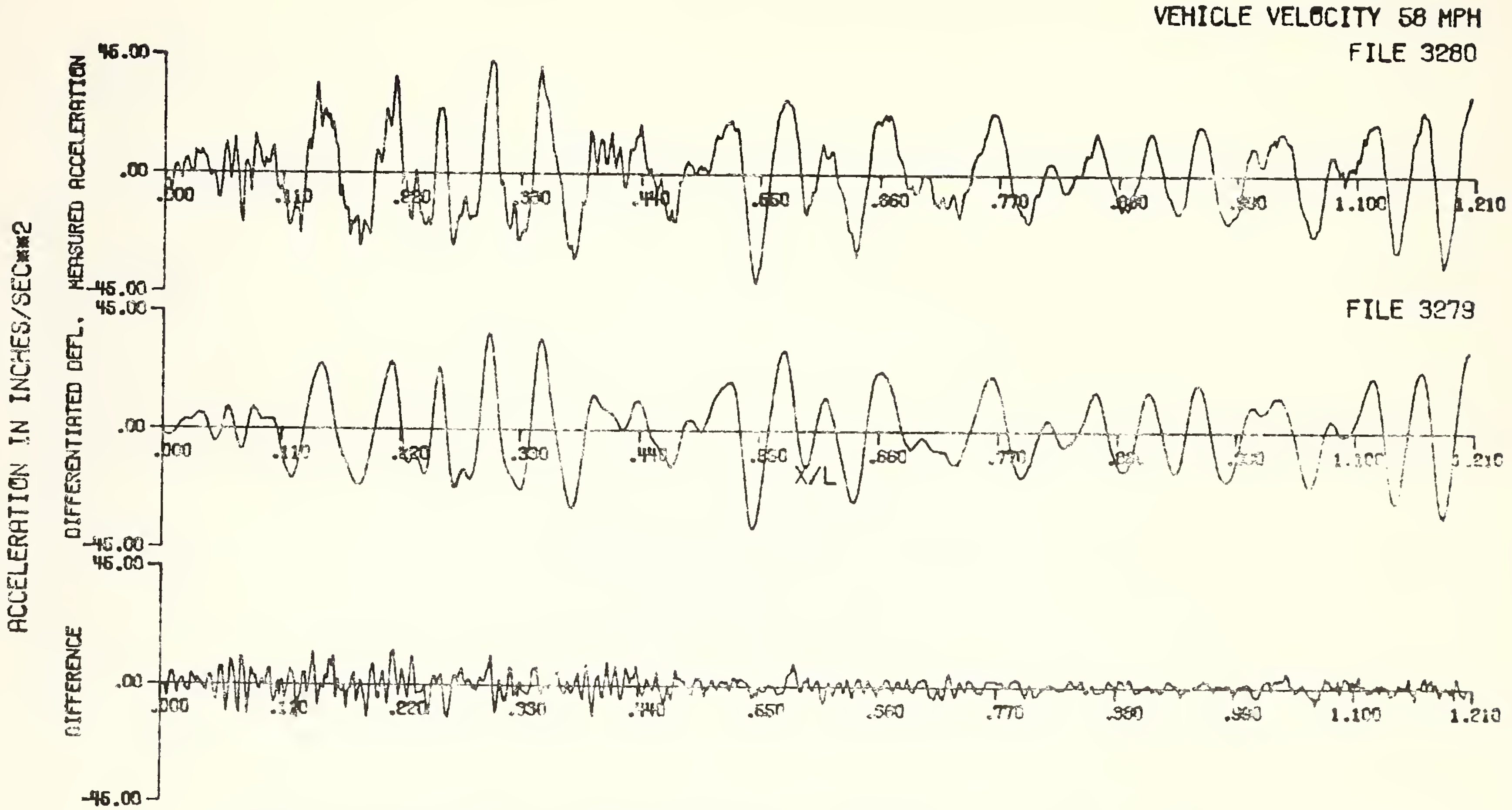


FIGURE 3.12. COMPARISON OF TWICE DIFFERENTIATED DEFLECTION RECORD WITH THE ACTUAL ACCELERATION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

3-SPAN CONTINUOUS STEEL BEAM BRIDGE
SPAN LENGTHS 68.0FT.85.0FT.68.0FT
WIDTH - 31.5FT
US 41(SB) OVER IROQUOIS RIVER NEWTON COUNTY
BRIDGE STUDY NUMBER CSB-C-1

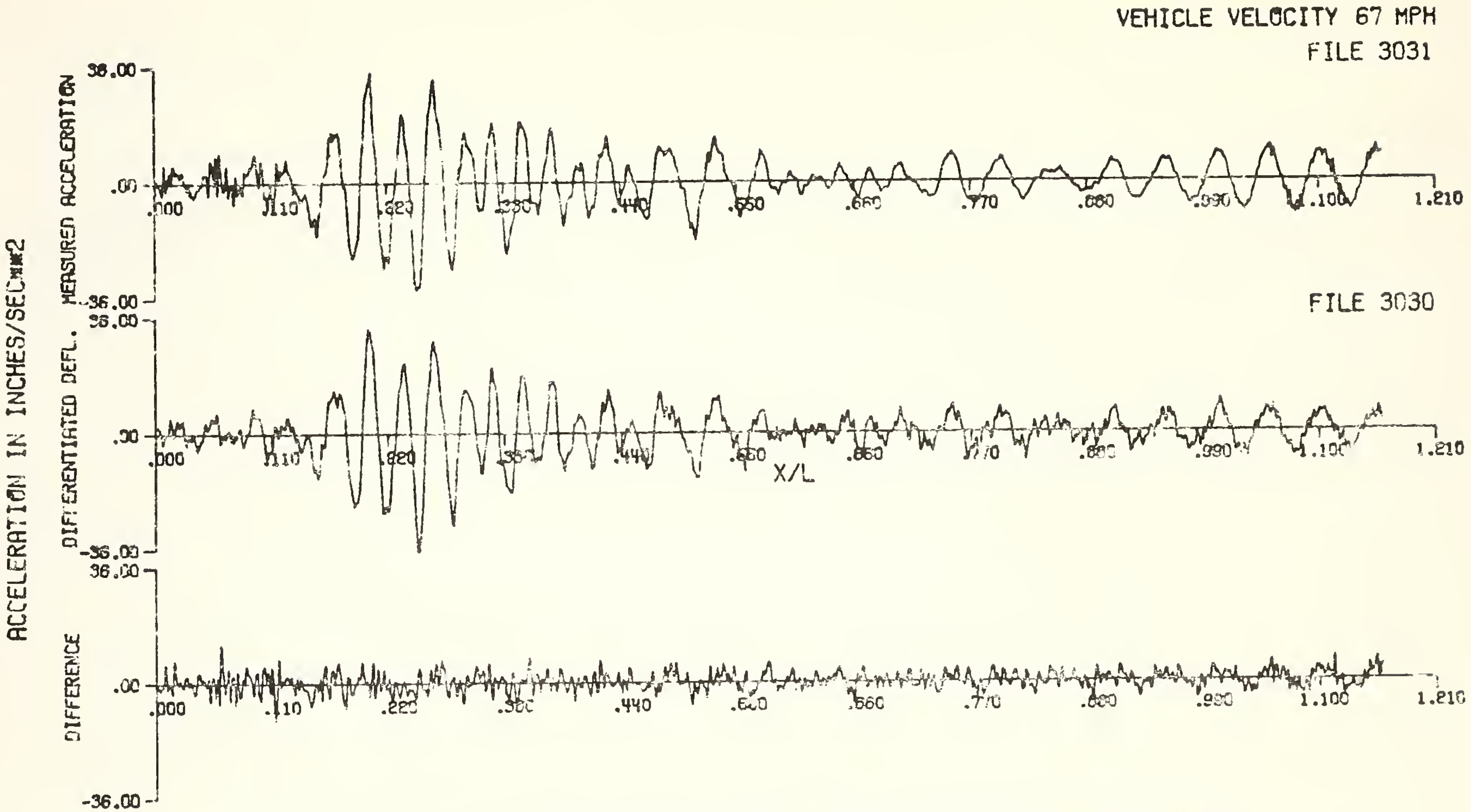


FIGURE 3.13. COMPARISON OF TWICE DIFFERENTIATED DEFLECTION RECORD WITH THE ACTUAL ACCELERATION RECORD
ACCELEROMETER AND DEFLECTION TRANSDUCER LOCATED AT SAME POSITION IN THE FIRST SPAN

4-SPAN PRESTRESSED CONCRETE I-BEAM BRIDGE
SPAN LENGTHS 70.75FT.72.0FT.72.0FT.70.75FT
WIDTH - 30.0FT
US 31(NB) OVER EEL RIVER MIAMI COUNTY
BRIDGE STUDY NUMBER PCIB-A-2

(3) smooth the deflection file and differentiate it either once, twice or three times for the corresponding velocity, acceleration and jerk, respectively,

(4) smooth the acceleration file (from Accelerometer #1 only) and calculate the corresponding jerk by differentiation,

(5) integrate the acceleration file either once or twice to obtain the corresponding velocity or deflection file,

(6) compare the deflection from the twice integrated acceleration with the actual deflection,

(7) compare the acceleration from the twice differentiated deflection with the actual acceleration,

(8) calculate the maximum values for deflection, velocity, acceleration and jerk,

(9) calculate the vehicle velocity and identify vehicle type,

(10) plot the deflection, velocity, acceleration, or jerk file for the portion of the file when the vehicle is on the bridge,

(11) plot the actual deflection versus the deflection from twice integrated acceleration,

(12) plot the actual acceleration and acceleration from twice differentiated and the difference between the two quantities,

(13) plot the acceleration files and their difference from two accelerometers located in the same span but on opposite sides of the bridge.

All of the 900 vehicle crossings digitized were reduced using the applicable portions of the computer programs described above and the results will be presented in the following chapter.

For a more detailed description of the computer programs and their operation see Appendix F.

3.3.2. Frequency Content of the Data

The frequency content of the deflection and acceleration files is important for several reasons. If analytical techniques are to provide consistent, accurate results, the modes of vibration which are excited as a vehicle traverses a bridge must be known. For example, the analytical computer program for two and three span bridges used by Shahabadi(1) considers the bridge structure as a single beam, thus ignoring the effects of the torsional modes. Therefore, if torsional modes are actually present, how much do these modes contribute to the total motion of the bridge? In the same vein, the frequency content or spectrum for the vibration of a certain bridge can be used to estimate the contribution of each dominant frequency, whether it be a torsional or bending frequency, to the total motion of the bridge structure. The frequency spectrum may also be used to calculate the actual stiffness of a bridge which in turn may be used to identify the amount of composite action present.

Frequency Response Analysis

The frequency content of a particular deflection or acceleration file is presented in a graphical form known as a Fourier spectrum. The ordinate of the discrete Fourier spectrum is the modulus of the discrete Fourier transform X_k in units of the function transformed and the abscissa is the corresponding frequency in Hertz (Hz). The Fourier transform X_k of a sample time history x_n is given by

$$X_k = \Delta t \sum_{n=0}^{N-1} x_n \exp\left[-j\,\frac{2\pi kn}{N}\right] \quad k = 0,1,\ldots,N/2 \qquad (3.1)$$

where the time history X_n is transformed from the time domain to the frequency domain.

Suppose x_n represents an acceleration file which was sampled at N equally spaced points at a distance of Δt seconds apart. The Fourier spectrum of this acceleration file would be a plot of the magnitude or modulus $|X_k|$ of the Fourier transform X_k in inches/second^2 versus the frequency content in Hz. Suppose also that the acceleration file under consideration was recorded from a bridge which only vibrates in its first two natural bending modes. Thus the Fourier spectrum would contain a definite peak or spike at each of these natural frequencies. In addition, this spectrum contains information concerning the magnitude of the acceleration file which was transformed. Suppose one of the frequency peaks f_n is comprised of 3 discrete moduli $|X_k|$, $k = m-1, m, m+1$ where $|X_{m-1}| < |X_m| > |X_{m+1}|$. An estimate of the mean square value of this f_n Hz. signal contained in the discrete acceleration history is

$$\overline{\psi}^2_{f_n} = 1/2 \sum_{k=m-1}^{m+1} |X_k|^2 \qquad (3.2)$$

In general, the mean square value is an indication of the magnitude of the f_n Hz. portion of the discrete time history. An obvious extension of Eq. (3.2) is the estimate of the mean square value of the entire time history, x_n, which becomes

$$\overline{\psi}^2_X = |X_o|^2 + |X_{N/2}|^2 + 1/2 \sum_{k=1}^{(N/2)-1} |X_k|^2 \qquad (3.3)$$

Appendix G contains a detailed explanation of the Fourier spectrum and its applications. An example problem is also included to provide further understanding into the utilization of the spectrum.

Frequency Spectrum Computer Programs

There is a separate program for calculating the Fourier spectrum for Phase I and Phase II data, with the main difference between the two programs being the calculation of vehicle types and lengths. Each program will calculate the Fourier moduli $|X_k|$ and their corresponding frequencies for each time history and then plot the Fourier spectrum.

Each data file, whether it be a deflection or acceleration file, may be divided into two parts. The first portion of each file is referred to as the vehicle crossing portion of the file and the second part, after the vehicle leaves the bridge, is referred to as the free vibration portion of the file. Each file is so divided in order to more readily identify which natural frequencies were actually excited as the vehicle crosses the bridge. It was found that the spectrum of the total file contained frequencies not directly associated with the motion of the bridge such as the effect of tire bounce and noise of the engine exhaust. These additional frequencies tended to reduce the sharpness of the frequency spectrum, thus obscuring the natural frequencies of the bridge. The computer program will calculate the Fourier spectrum for either the vehicle crossing or free vibration portion of each file with the spectrum for the free vibration of the bridge structure containing only the natural frequencies which were excited by the vehicle crossing. For a further description of the frequency computer program see Section E.3 of Appendix E.

CHAPTER IV: SUMMARY OF TEST RESULTS

4.1 General

This chapter will contain the results of the Phase I and Phase II test program. The basic response characteristics for each type of bridge structure will be presented followed by an investigation of the effects of the major bridge and vehicle parameters on the response characteristics. The major bridge and vehicle parameters are bridge type, vehicle size and velocity, and the transverse location of the vehicle on the bridge, while the response characteristics include the maximum values of displacement, velocity, acceleration, and jerk together with the frequency content. Finally, the frequency content will be analyzed and presented. The following section contains a description of the procedures used to obtain the various response characteristics excluding the frequency analysis procedure which was described in Appendix G.

4.2 Analysis Procedures

Each vehicle crossing record was analyzed using the computer program and techniques presented in Appendix F and furnished the following response characteristics:

(1) maximum displacement
(2) maximum acceleration from each accelerometer
(3) damping ratio
(4) maximum velocity and displacement from integrated acceleration

(5) maximum velocities, accelerations, and jerks from differentiated displacement, smoothed using a 3rd degree polynomial with K=3 through K=15 points

(6) maximum jerk from differentiated acceleration, smoothed with a 3rd degree polynomial and K =3 through K=15 points.

For the sake of clarity, a 3rd degree polynomial will be referred to as J = 3 and the number of data points used in the smoothing will be (2 x K) + 1. K = 3 will signify that seven consecutive data points were least squares fit to the specified polynomial with 3 (K) data points either side of the point under consideration.

The maximum displacement reported (minus (-) down and plus (+) up) is the largest absolute value excited as a vehicle traverses the bridge. The maximum acceleration presented is also the largest absolute value encountered in the record and will always be considered positive.

The maximum velocity and jerk were found by two different procedures and the results averaged. The displacement was smoothed with J = 3 and K = 3, 15 and differentiated twice to obtain acceleration. Two criteria were used to determine which smoothing polynomial most closely approximated the measured acceleration the first of which was calculating the variance between the measured and the calculated acceleration files. The second criteria was comparing the maximum acceleration for each of the 13 smoothing polynomials with the maximum measured acceleration. A range of smoothing polynomials which most consistently gave results which enveloped the maximum measured acceleration were then selected. It should be noted that the polynomial which resulted in the lowest variance smoothed the displacement more than the smoothing polynomial which resulted in the closest maximum acceleration. This fact

was also taken into consideration when selecting the most efficient polynomial. After this range of smoothing polynomials had been selected, the maximum velocity and jerk from displacement could be calculated using the same range for both quantities and resulted in two maximum values for each quantity corresponding to the low and high value of the selected range.

The third maximum velocity value was obtained by integrating the acceleration record using the techniques developed in Appendix F. Therefore, the maximum velocity which will be presented later in this chapter is the average of three maximum values, two from the displacement and one from acceleration.

The maximum value of jerk was obtained in a slightly different manner. For the range of smoothing polynomial J = 3, K = 3, 15 the maximum differentiated acceleration (jerk) varied by a factor of 15 to 30. Since the only other values of maximum jerk available are those from the displacement, the maximum jerk from acceleration was chosen by selecting a range of smoothing polynomials which would consistently envelope the range of maximum jerk values from displacement. Therefore, the maximum reported jerk values will be the average of four maximums, two from displacement and two from acceleration.

In summary, a range of smoothing polynomials were chosen which, when applied to the displacement file, would result in the best approximation of the corresponding acceleration file. This range of polynomials would then be used to obtain a range of maximum velocity and jerk values from displacement. The jerk values chosen would then be used to determine what range of smoothing polynomials had to be applied to the acceleration file to produce maximum jerk values consistent with those from displacement.

Table 4.1. Most Efficient Smoothing Used in Differentiating the Displacement and Acceleration to Calculate the Corresponding Acceleration and Jerk

3rd Degree Polynomial Using K Points

Bridge Type	K for Acceleration from Displacement	K for Jerk from Acceleration
SB-A-4	3,4	4,6
SB-B-1	4,5	3,5
SB-C-1	3,6	3,6
	5	6
KCSB-B-1	4	4,5
KCPG-A-1	5,6	3,6
CSB-A-2	3	3
CSB-B-1	5,7	3,8
CSB-C-1	5,7	3,7
	3,5	3,6
RCG-A-3	3,4	3,4
RCG-B-3	4,6	3,5
CRCS-A-1	4,5	3,5
CRCS-A-5	3,4	3,5
PCIB-A-1	5,6	3,8
PCIB-A-2	4	3

Table 4.1 summarizes the smoothing results for a representative sample of the 62 bridges tested. The results from two different tests on the same bridge are included to show there is some variability between tests on the same bridge as well as between bridge types. In the cases where K is a single value the calculated acceleration or jerk was consistently close to the actual value and no range of values was required. In summary, for all types of bridges and vehicles the maximum acceleration may be obtained by averaging the maximum values from smoothing the displacement with J = 3, K = 4, 5 and differentiating twice. The maximum jerk may be obtained by smoothing the acceleration with J = 3, K = 3,6 and averaging the results after differentiation.

4.3. Results of the Testing Program

This section will summarize all of the data collected and analyzed during both Phase I and Phase II testing. The results are presented in Table 4.2 and contain the mean, sample standard deviation and range for each of the following quantities: the vehicle type and velocity; maximum displacement; velocity; acceleration; and jerk for each subcategory of bridges listed in Appendix A. The bridges in Category 3 - Subcategory A, Category 6 - Subcategory B, and Category 8 are not included in Table 4.2 since there was not a sufficient amount of data available to form any conclusions.

Since the vehicle classification codes (see Table 2.2) do not represent the relative weights of the vehicles, a vehicle weight factor was used and is shown and compared with its corresponding vehicle classification code in Table 4.3. The vehicle weight factor is not intended to represent the true weight relationship between different vehicle types but is used to give a general idea of the

Table 4.2. Summary of the Results of the Testing Program

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velocity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Single Span	5.7	46	-0.007	0.65	31	3870
SB-A-1	2.8	15	0.019	0.38	24	2100
thru	9	76	-0.060	1.38	89	7760
SB-A-5	1	18	+0.046	0.20	12	1210
Single Span	4.0	35	-0.014	0.39	18	1820
SB-B-1	1.9	12	0.010	0.20	12	990
thru	9	61	-0.047	0.99	57	3950
SB-B-3	1	15	+0.008	0.13	6	440
Single Span	5.1	41	-0.025	0.37	15	810
SB-C-1	2.7	10	0.010	0.25	12	530
	9	57	-0.058	1.17	50	2320
	1	20	-0.006	0.08	3	170
Two Span	3.5	35	-0.082	NA	15	NA
KCSB-A-1	1.3	11	0.067		7	
and	5	52	-0.277		34	
KCSB-A-2	1	19	-0.004		3	
Two Span	3.6	36	-0.107	0.64	25	2130
KCSB-B-1	1.0	6	0.030	0.19	7	640
	4	43	-0.124	0.89	33	2650
	1	26	-0.024	0.13	8	740

Table 4.2. Continued

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velocity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Two Span	7.3	56	-0.104	NA	63	NA
KCSB-C-1	2.7	6	0.087		36	
thru	9	64	-0.228		133	
KCSB-C-4	3	45	+0.040		21	
Two Span	3.9	30	-0.086	0.63	16	550
KCSB-D-1	0.4	8	0.028	0.30	10	430
and	4	45	-0.123	1.2	42	1680
KCSB-D-2	2	15	-0.031	0.10	2	75
Two Span	3.8	30	-0.109	0.48	17	220
KCSG-A-1	0.5	9	0.030	0.17	6	60
and	4	50	-0.131	0.71	30	320
KCSG-A-2	2	17	-0.030	0.14	4	83
Two Span	4.0	32	-0.081	NA	18	NA
KCSG-B-1	0	10	0.032		3	
and	4	51	-0.117		23	
KCSG-B-2	4	18	-0.044		13	
Two Span	3.9	33	-0.075	0.43	17	720
KCPG-A-1	0.4	10	0.027	0.15	6	280
thru	4	49	-0.112	0.59	27	1190
KCPG-A-3	3	18	-0.019	0.11	5	350

Table 4.2. Continued

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velocity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Two Span	3.8	33	-0.073	NA	17	NA
KCPG-B-1	0.7	9	0.041		7	
and	4	49	-0.159		36	
KCPG-B-2	1	17	-0.023		6	
Three Span	7.4	53	-0.002	0.71	51	4840
CSB-A-1	2.4	5	0.033	0.31	20	1940
thru	9	62	-0.049	1.55	105	7790
CSB-A-4	1	41	+0.143	0.29	24	1620
Three Span	6.4	55	-0.024	0.69	42	2150
CSB-B-1	2.9	11	0.027	0.33	25	970
thru	9	75	-0.075	1.40	124	4580
CSB-B-4	1	21	+0.066	0.15	10	450
Three Span	6.9	53	-0.055	0.87	40	2100
CSB-C-1	2.6	9	0.048	0.47	19	1900
	9	67	-0.137	1.84	83	5910
	2	28	+0.051	0.07	12	250
Three Span	7.0	53	-0.005	0.20	21	2060
RCG-A-1	2.7	6	0.003	0.12	9	1360
thru	9	64	-0.014	0.43	45	4300
RCG-A-3	1	39	+0.001	0.05	4	330

Table 4.2. Continued

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velocity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Three Span	7.3	56	-0.006	0.13	20	850
RCG-B-1	2.7	9	0.006	0.04	11	280
thru	9	66	-0.025	0.21	59	1440
RCG-B-4	1	20	+0.008	0.04	2	190
Three Span	8.2	61	-0.002	0.09	12	640
CRCS-A-1	2.0	7	0.004	0.03	6	260
thru	9	76	-0.008	0.15	30	1280
CRCS-A-6	1	40	-0.001	0.03	3	100
Three Span	5.4	52	+0.004	NA	42	NA
PCIB-B-1	3.3	11	0.016		32	
	9	68	-0.013		90	
	2	40	+0.027		21	
Four Span	5.3	52	-0.023	0.38	20	1417
PCIB-A-1	2.7	9	0.015	0.21	10	490
and	9	70	-0.066	0.89	41	3280
PCIB-A-2	1	32	-0.005	0.07	3	300

Note: The values for each bridge or vehicle parameter are the following: mean value, standard deviation, and range respectively.

Table 4.3. Relationship Between the Vehicle Weight Factor and the Vehicle Classification Code

	Vehicle Weight Factor	Vehicle Class. Code	Vehicle Description
Phase I			
	1	1	Passenger Car
	2	2	Pick-ups - 2 ton trucks
	4	4	R & TC Bus
	9	3	Heavy trucks
Phase II			
	1	1	Passenger Car
	2	3	Pick-up/Vans
	3	4	2-axle truck
	4	2	R & TC Bus
	5	5	3-axle truck
	6	6	2-axle tractor/1-axle trailer
	7	7	2-axle tractor/2-axle trailer
	8	8	3-axle tractor/1-axle trailer
	9	9	3-axle tractor/2-axle trailer

types and weights of vehicles used in the testing of a bridge. However, the weight factors are arranged in ascending order according to the expected vehicle weight.

The values of maximum displacement, velocity, acceleration, and jerk presented in Table 4.2 were determined by the methods presented in the previous section. It should be noted that the maximum mean displacement was calculated using the appropriate sign of the displacement since the upward maximum deflection was larger than the downward maximum deflection for some cases. Also the maximum displacement, velocity, and jerk are for only one point on the bridge, while the maximum acceleration was the maximum value of the output from all accelerometers on the bridge deck. It was found though, that the maximum acceleration never occurred consistently at one particular position on any of the bridges tested.

The results presented in Table 4.2 are for a wide range of weight factors and vehicle velocities which makes the comparison of various properties between bridge categories difficult. However, in most cases this has one advantage in that the results for each subcategory of bridges are representative of the response of the bridges under the actual traffic experienced by the bridges. A good example of this fact is the two span bridges in Categories 5 and 6 (see Appendix A) which are all located on county roads. The mean acceleration of these bridges is 17-18 in/sec^2 with a vehicle weight factor of 4.0 which corresponding to test vehicle. These bridges will probably never see much larger or heavier vehicles than the test bus. Thus, the mean acceleration presented will be representative for that type of bridge.

On bridges which do carry heavy truck traffic we are interested in the maximum response properties. Table 4.4 presents the results of Table 4.2 but only for the heaviest

Table 4.4. Summary of the Results of the Testing Program for Vehicles with a Weight Factor of 9

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velocity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Single Span	9	54	-0.006	0.87	61	5080
SB-A-1		15	0.026	0.36	15	2240
thru		76	-0.060	1.38	89	7760
SB-A-5		33	+0.046	0.59	41	3180
Single Span	9	47	-0.031	0.67	33	1490
SB-C-1		7	0.013	0.29	11	520
		57	-0.058	1.16	50	2320
		36	-0.018	0.30	18	870
Two Span	9	58	-0.089	NA	74	NA
KCSB-C-1		5	0.093		35	
thru		64	-0.228		133	
KCSB-C-4		51	+0.040		33	
Three Span	9	53	-0.003	0.82	53	5110
CSB-A-1		5	0.041	0.34	18	1700
thru		60	-0.049	1.55	105	7790
CSB-A-4		41	+0.143	0.39	31	2520
Three Span	9	60	-0.032	0.97	58	2900
CSB-B-1		5	0.032	0.17	19	680
thru		69	-0.075	1.40	123	4580
CSB-B-4		48	+0.066	0.81	30	2160

Table 4.4. Continued

Bridge Type	Vehicle Weight Factor	Vehicle Velocity (MPH)	Maximum Displacement (inches)	Maximum Velcoity (in/sec)	Maximum Acceleration (in/sec^2)	Maximum Jerk (in/sec^3)
Three Span	9	57	-0.068	1.20	52	2740
CSB-C-1		4	0.059	0.33	13	930
		67	-0.137	1.84	83	5910
		49	+0.051	0.70	29	1500
Three Span	9	54	-0.007	0.29	24	3150
RCB-A-1		5	0.003	0.11	9	1170
thru		64	-0.014	0.43	45	4300
RCG-A-3		48	-0.003	0.14	17	1400
Three Span	9	60	-0.009	0.15	25	1150
RCB-B-1		5	0.006	0.04	10	200
thru		66	-0.025	0.21	59	1440
RCG-B-4		49	-0.003	0.12	14	930
Three Span	9	62	-0.003	0.10	16	700
CRCS-A-1		6	0.004	0.02	6	220
thru		76	-0.008	0.15	30	1280
CRCS-A-6		50	-0.002	0.07	8	390
Four Span	9	61	-0.037	0.61	31	2180
PCIB-A-1		6	0.014	0.16	6	540
and		70	-0.066	0.89	41	3280
PCIB-A-2		50	-0.018	0.44	20	1340

Note: The values for each bridge or vehicle parameter are the following: mean value, standard deviation, and range respectively.

vehicle (9). Only bridges having a significant amount of weight factor 9 vehicles are included. In terms of mean acceleration, all bridges of reinforced concrete construction (Categories 7, 9, 10) have approximately half the acceleration level as experienced by steel beam and plate girder construction. Among bridges of steel construction the short single span (SB-A) and two span structures (KCSB-C) have higher mean values of acceleration than do the three span bridges, although the maximum accelerations for all these bridges fall within the same range. In general, the mean jerk seems to be slightly higher for steel than for concrete bridges but this trend is not as apparent as it was for acceleration. The major trend of jerk is the fact that the jerk increases considerably with the decrease in span length for a certain type of bridge. The mean jerk is 5080 in/sec^3 for a 60 foot span SB-A and 1490 in/sec^3 for a 72 foot SB-C single span. For 47'-57'-47' CSB-A, 60'-72'-60' CSB-B and 68'-85'-68' CSB-C three span bridges, the mean jerk is 5110, 2900, and 2740 in/sec^3 respectively. The only evident trend in the bridge velocity is that the values are lower for reinforced concrete bridges than for steel beam bridges.

Since steel beam or plate girder bridges have generally higher response values, a summary of the mean response values for all bridges of steel construction are shown in Table 4.5. Single through three span bridges in order of increasing span length are listed. The mean displacement was omitted, since it would not present any pertinent information. Several of the trends mentioned in the previous paragraph may be seen with some exceptions. One of the exceptions is the mean acceleration and jerk for the 55 foot single span SB-B. If these quantities followed previous trends they should be higher than the values for the 60 foot single span SB-A. The reason for

Table 4.5. Summary of the Mean Values from Table 4.2 for Steel Beam or Plate Girder Bridges and One Through Three Spans

Bridge Type	Span Lengths (feet)	Mean Weight Factor	Mean Vehicle Vel. (MPH)	Mean Velocity (in/sec)	Mean Acceleration (in/sec^2)	Mean Jerk (in/sec^3)
SB-B	55'	4.0	35	0.39	18	1820
SB-A	60'	5.7	46	0.65	31	3870
SB-C	72'	5.1	41	0.37	15	810
KCSB-C	76'-76'	7.3	56	NA	63	NA
KCSB-D	96'-96'	3.9	30	0.63	16	550
KCSB-A	98'-98'	3.5	35	NA	15	NA
KCPG-B	102.5'-102.5'	3.8	33	NA	17	NA
KCSB-B	103.5'-103.5'	3.6	36	0.64	25	2130
KCSG-A	122.5'-122.5'	3.8	30	0.48	17	220
KCSG-B	128'-128'	4.0	32	NA	18	NA
KCPG-A	129'-129'	3.9	33	0.43	17	720
CSB-A	47.5'-57'-47.5'	7.4	53	0.71	51	4840
CSB-B	60'-72'-60'	6.4	55	0.69	42	2150
CSB-C	68'-85'-68'	6.9	53	0.87	40	2100

this discrepancy is the lack of heavy truck traffic, as can be seen by comparing the mean vehicle weight factors for the two bridges.

It should be noted that nothing has been said up to this point about bridge and vehicle variables over which there was no control or knowledge. For instance, the results of Aramraks (2) acceleration study indicated that bridge roughness played a major role in the magnitude of acceleration experienced by a bridge. Also, the approach profile and various vehicle parameters are unknown and have not been considered. Even with the vehicle classification code (see Table 2.2), the weight of a number 9 vehicle might vary as much as 260%, which directly affects the acceleration. This fact may be seen by examining the range of response properties for number 9 vehicles in Table 4.4. These facts do not detract from the value of the test program, since the main objective was to ascertain the general response characteristics which was done and which are presented in Table 4.2. Shahabadi(1) does investigate the deck roughness for six of the bridges included in this test program and presents a technique where the surface roughness is approximated by a random generated function.

However, the special tests described in Chapter 2 made it possible to investigate the effect of vehicle velocity on displacement, velocity, acceleration and jerk. The results are shown in Figures 4.1 through 4.4, respectively, with each figure containing the results from one single span, one two span, and one three span bridge. In all cases the vehicle used to excite the bridge was the Research and Training Center Bus crossing the bridge in the travel lane which is on the same side of the bridge as the deflection gage. Figure 4.1 presents the maximum displacement versus the vehicle velocity. The discrete

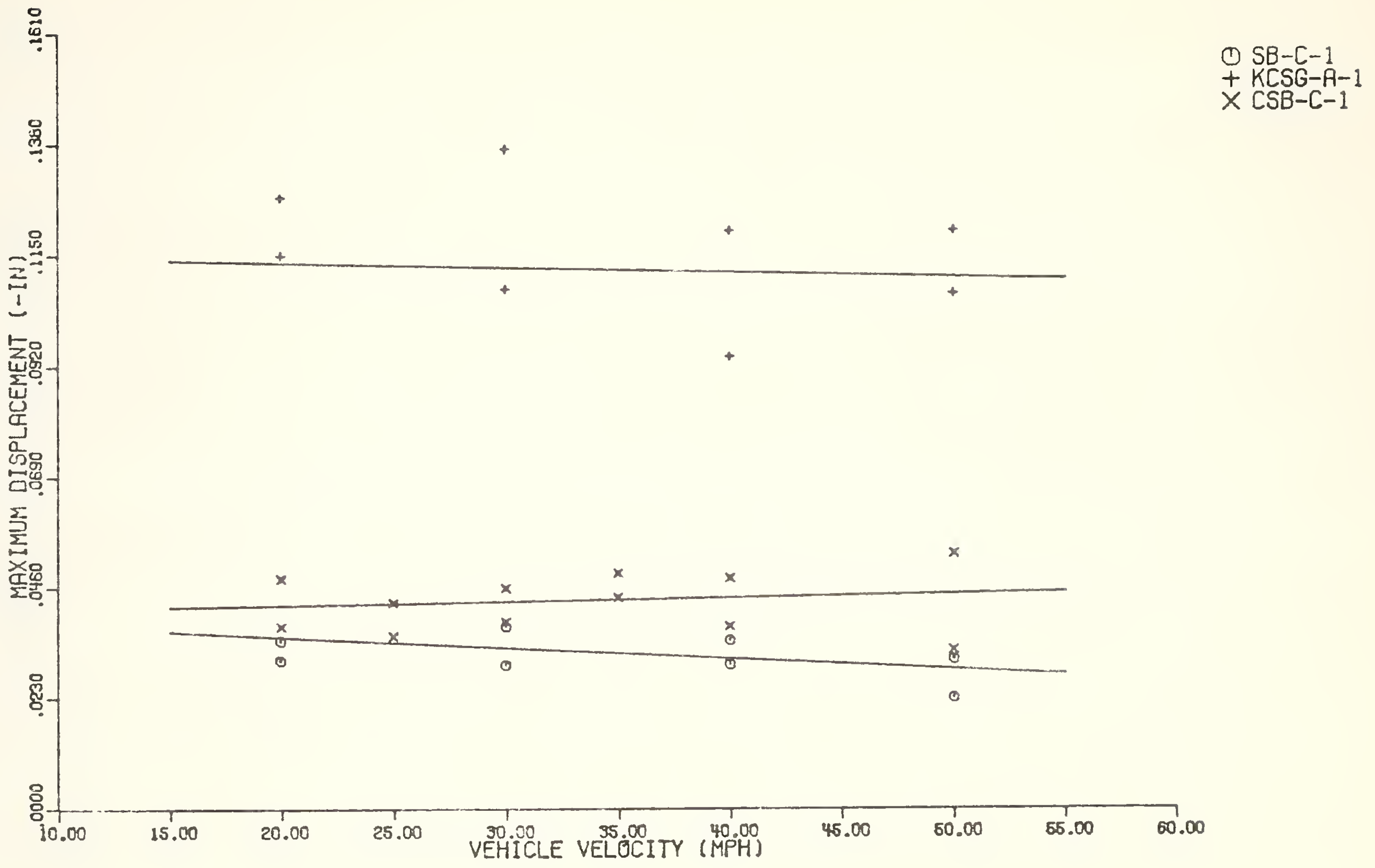


Figure 4.1. Maximum Displacement versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus.

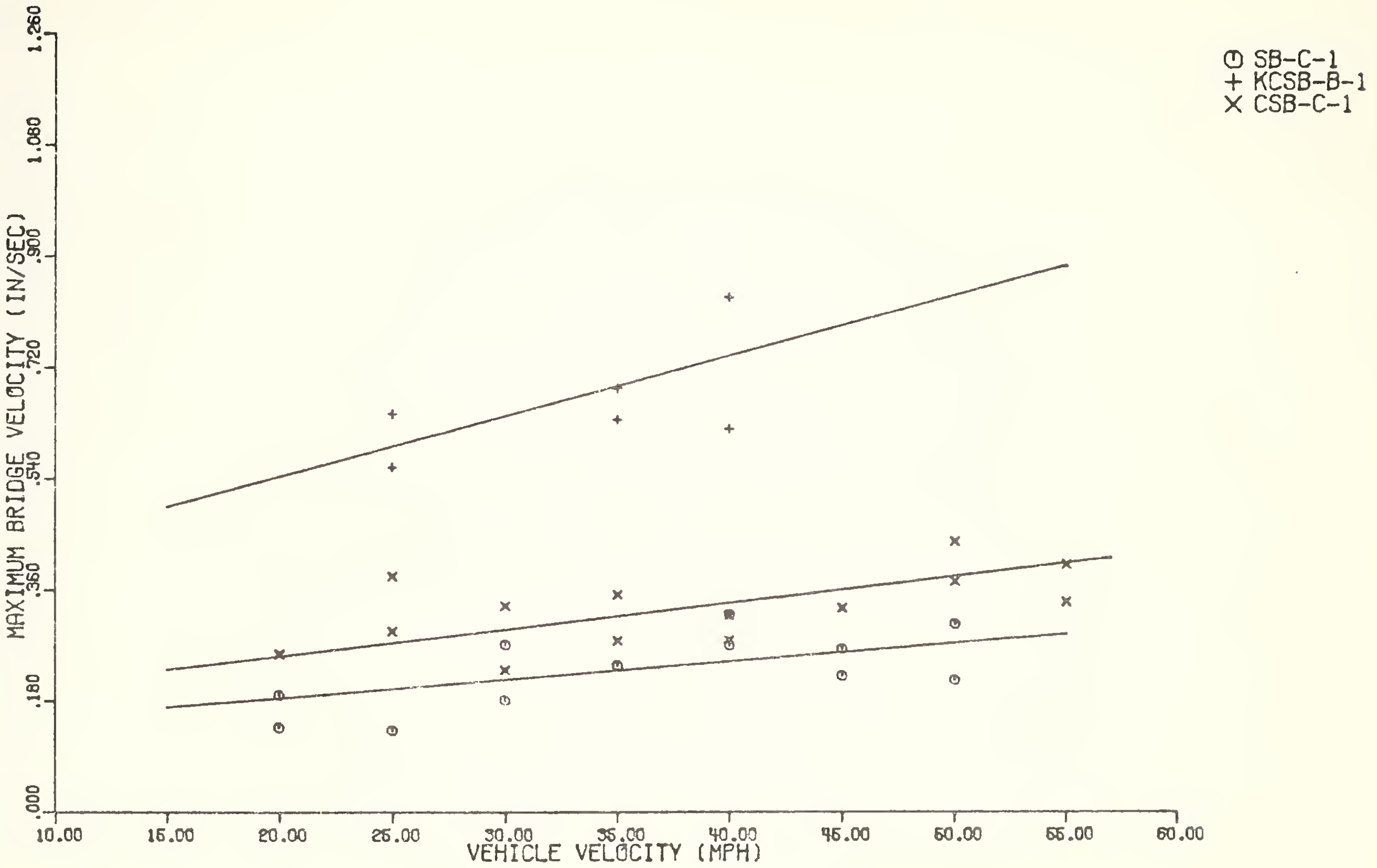


Figure 4.2. Maximum Bridge Velocity versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus

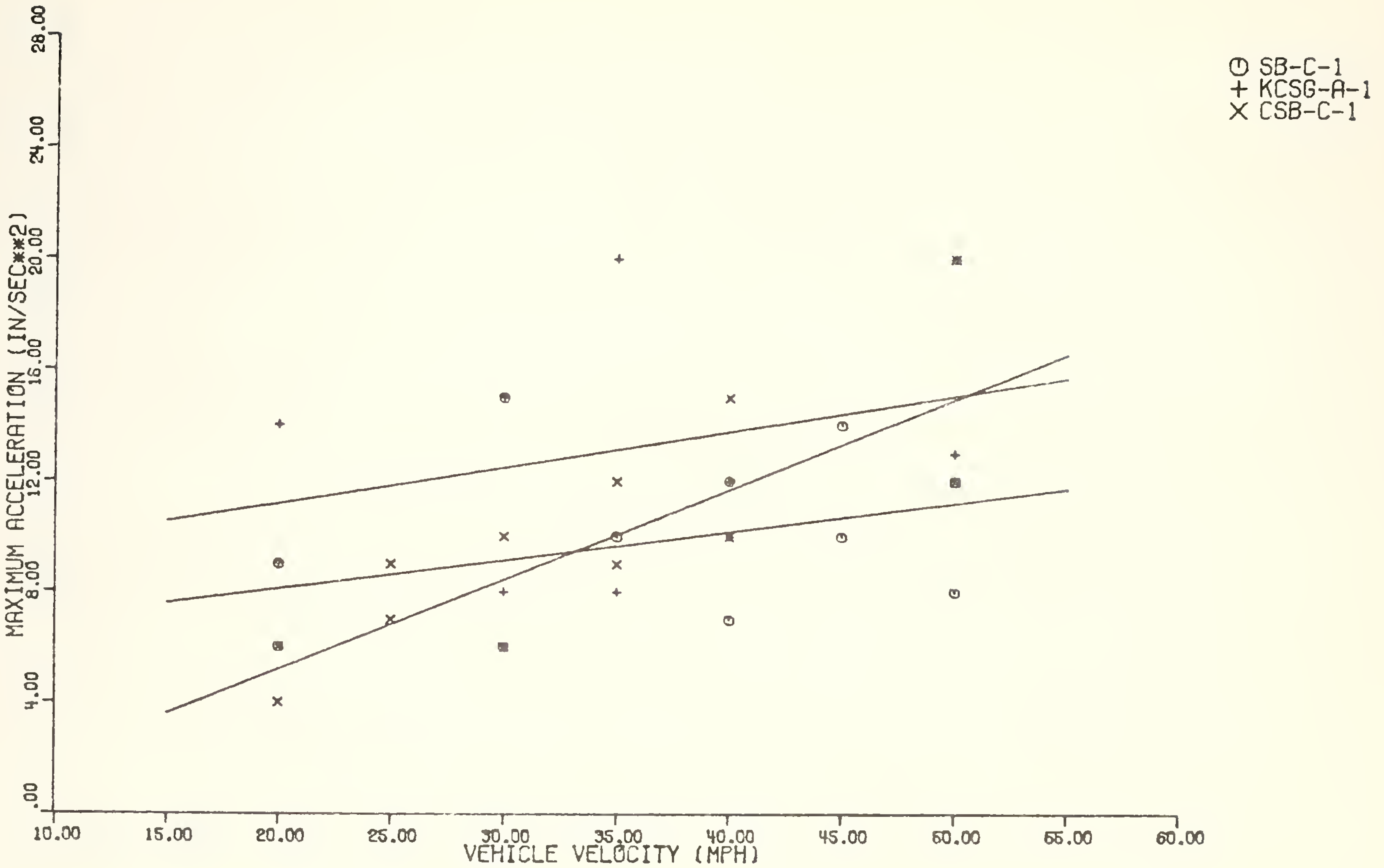


Figure 4.3. Maximum Acceleration versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus.

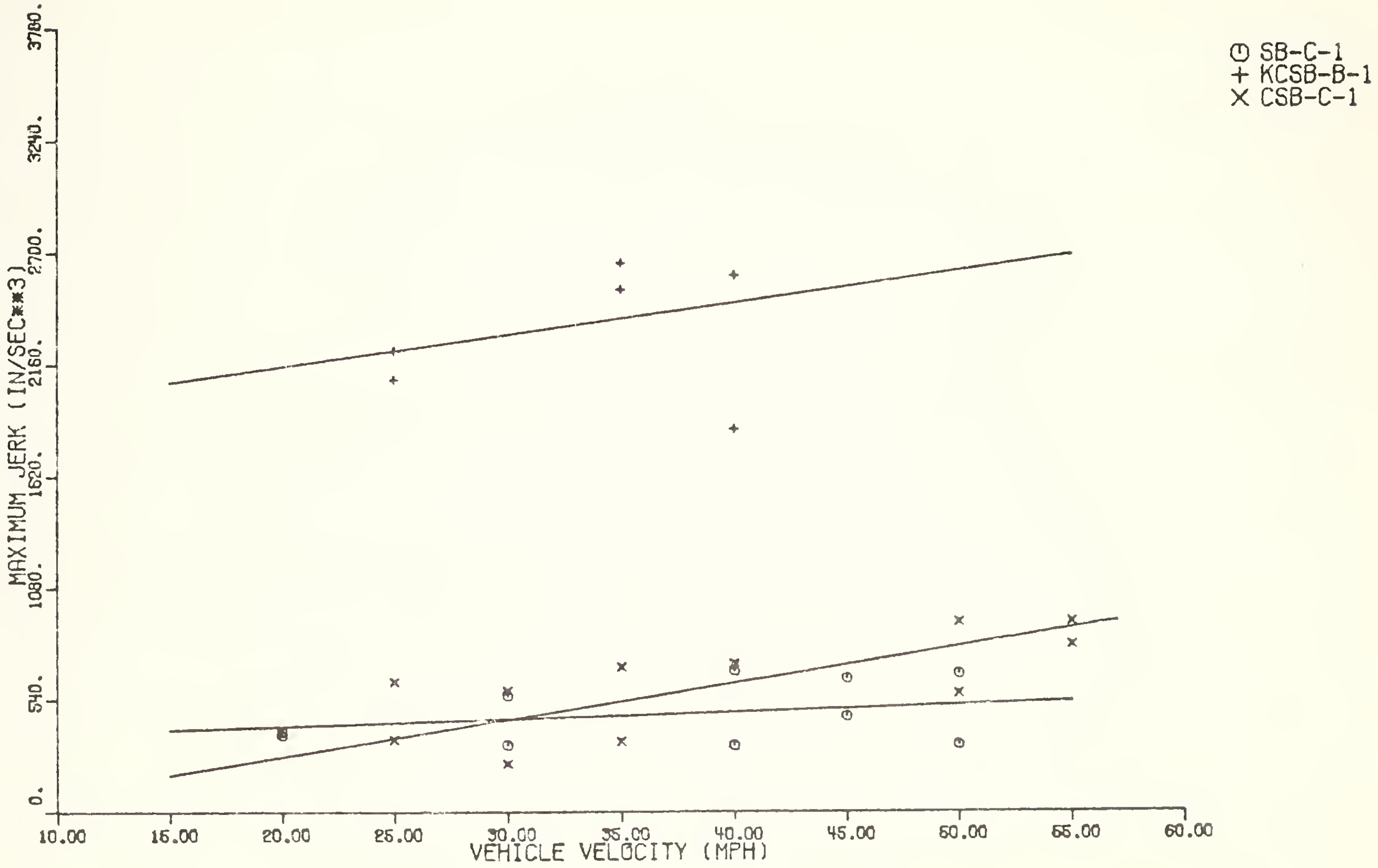


Figure 4.4. Maximum Jerk versus Vehicle Velocity for One through Three Span Bridges--Vehicle, R & TC Bus.

data values plotted are the range of values for that particular velocity and are shown since to plot all data points would be prohibitive. All data are linear least squares fit and the results plotted as the solid line. The two span KCSG-A-1 shows a very slight decrease and the three span CSB-C-1 a very slight increase in displacement as the vehicle velocity increases while the results from the single span bridge show a marked decrease with increasing speed.

Figures 4.2, 4.3, 4.4 indicated that bridge velocity, acceleration and jerk all increase with increasing vehicle velocity which was to be expected. It is interesting to note that the magnitude of jerk values for KCSB-B-1 in Figure 4.4 are considerably higher than the single and three span bridges. This fact also can be seen in Table 4.5 where the mean acceleration and jerk are substantially higher than other comparable two span bridges with the same vehicle weight factor. This might be do in part to the deck roughness and/or approach slab conditions mentioned earlier.

One of the significant findings of this investigation is the magnitude of acceleration actually experienced by the bridges. The highest acceleration experienced by any of the bridges was 133 in/sec^2, while the highest mean acceleration was 74 in/sec^2 and there were only five instances where the maximum acceleration for a particular vehicle crossing was larger than 100 in/sec^2. Although the results of this test program are more thoroughly analyzed by Shahabadi(1) in light of the human comfort criteria, it would be interesting to note that the magnitude of the accelerations experienced by all the bridges in this test program fall between the "imperceptible" to "unpleasant to a few" categories of the acceleration criteria for human response to harmonic vertical motion(26) presented by Wright and Walker and shown in Table 4.6.

Table 4.6. Acceleration Criteria for Human Response to Harmonic Vertical Vibration(26)

Human Response	Amplitude of Acceleration (in/sec^2) Transient	 Sustained
Imperceptible		
	5	0.5
Perceptible to few		
	10	1
Perceptible to some		
	20	2
Perceptible		
	50	5
Unpleasant to few		
	100	10
Unpleasant to some		
	200	20
Unpleasant		
	500	50
Intolerable to few		
	1000	100
Intolerable to some		
	2000	200
Intolerable		

Table 4.7. Typical Damping Ratios

Bridge Type	Damping Ratio
Single Span	
SB-A	2.0%
SB-B	2.2%
SB-C	1.4%
Two Span	
KCSB-A	1.1%
KCSB-B	1.1%
KCSB-C	1.5%
KCSB-D	1.4%
KCSG-A	1.2%
KCSG-B	1.8%
KCPG-A	1.2%
KCPG-B	1.2%
Three Span	
CSB-A	2.5%
CSB-B	1.8%
CSB-C	1.6%
Four Span	
PCIB-A	1.7%

The damping ratios for most sub-categories of bridges are shown in Table 4.7. For very stiff bridges the vibrations decayed too rapidly thus prohibiting the calculation of a good value for the damping ratio. The quantities shown in this Table agree with those obtained in previous investigations[16,24] although the value of 2.5% for the three-span (47.5', 57', 47.5') continuous steel beam bridges should be noted.

4.4. Results of the Frequency Analysis

This section will present the results of the frequency analysis. The methods and computer program used in this analysis are described and contained in Appendix G. Since space will not permit the presentation of the frequency content of all the data records collected, the Fourier spectrum for one bridge from each sub-category will be shown. For each of these bridges, the spectrum of the deflection and two acceleration files for one vehicle type are plotted and presented in Appendix H.

The results from the spectral analysis are summarized and shown in Table 4.8. To facilitate the identification of the fundamental bending frequency and to investigate the amount of composite action developed by the appropriate steel beam bridges, the theoretical fundamental bending frequency was calculated using the bridge section properties supplied by the test crew. For composite steel beam bridges and concrete bridges the stiffness was taken equal to the sum of effective EI's of the beams considering their tributary slabs. The non-composite stiffness was taken equal to the sum of the stiffnesses of the steel beams plus the stiffness of the slab, K_{slab} where

$$K_{slab} = \frac{E_c t^3 b}{12(1-\mu^2)} \tag{4.1}$$

Table 4.8. Summary of the Frequency Analysis

Bridge Type	Fundamental Bending Frequency (Hz)			Percent Composite Action	Measured Fundamental Torsional Frequency(Hz)	Additional Frequencies (Hz) Excited
	Theoretical					
	Non Composite	Composite	Measured			
Single Span						
SB-A-4	4.13	6.39	7.62	100%	8.40	9.96 14.4 22.5 23.1
SB-B-1	4.45	6.95	6.77/7.07	100%	7.88	10.4 11.2
SB-C-1	3.62	4.82	4.88	100%	5.27	7.32 12.0
Two Span						
KCSB-A-1		2.71	2.73		3.32	4.39 4.69
KCSB-B-1	NA	2.19	2.15		2.73	3.91 6.83 7.42 12.3 14.3
KCSB-C-2	NA	3.73	3.81		4.10	5.66 9.08 10.15
KCSB-D-2	NA	2.62	2.83		3.03	3.41 4.54 8.30 8.89 15.8 20.7
KCSG-A-1	NA	2.27	2.34		2.83	3.42 3.80 12.99 13.77
KCSG-B-1	NA	2.20	2.22		4.49	8.30 8.89 15.8 20.7
KCPG-A-1	NA	2.36	2.44		3.12	4.10 7.81 8.11 17.6
KCPG-B-2	NA	2.29	2.25		2.83	3.61 3.91

Table 4.8. Continued

Bridge Type	Fundamental Bending Frequency (Hz)			Percent Composite Action	Measured Fundamental Torsional Frequency(Hz)	Additional Frequencies (Hz) Excited
	Theoretical					
	Non Composite	Composite	Measured			
Three Span						
CSB-A-2	6.31	11.11	7.54	7%	8.96	10.3 14.1 15.9 20.8
CSB-B-1	4.96	8.13	5.27	1%	5.96	7.42 8.20 9.77 10.8 15.2
CSB-B-2	4.96	8.13	5.27	1%	5.96	7.42 9.77
CSB-C-1	3.88	6.48	3.91	0%	4.49	5.86
RCG-A-3	NA	15.43	13.5/16.7			11.0 thru 33.0 Hz
RCG-B-3	NA	13.57	12.0/18.0			20.2 24.0 28.7
CRCS-A-5	NA	14.69	15.0/24.0			
PCIB-B-1	NA	13.66	13.41			20.6 22.0 26.9
Four Span						
PCIB-A-1	NA	6.26	6.45		6.93	7.42 7.91 9.18 14.0
PCIB-A-2	NA	6.26	6.45		6.93	7.42 7.91 9.18 13.5

E_c = modulus of elasticity of the concrete slab
t = thickness of the slab
μ = Poisson's ratio (assumed as 0.3)
b = total width of slab.

E_c was calculated assuming a concrete strength of 4000 psi. The results will be discussed at the end of this section.

The measured frequencies presented in Table 4.8 summarize the predominant excited frequencies shown in Fourier spectra contained in Figures H.1a through H.21c. Again it should be noted that Appendix H contains only a small portion of the spectra used in obtaining the values presented in Table 4.8. The major portion of these spectra are for the free vibration portion of each file. This was done since the vehicle crossing portion of the each file usually contains frequencies associated with the vehicle traversing the bridge together with the fact that the frequency resolution is fairly coarse for the spectrum of the vehicle crossing portion of the file. This fact may be seen by comparing the vehicle crossing spectra Figures H.14a, H.14c, and H.14e with their corresponding free vibration spectra Figures H.14b, H.14d, and H.14f. It should be noted that the particular vehicle crossing the bridge in these figures is forcing the bridge to vibrate primarily in the 9.77 Hz and 15.2 Hz mode where the fundamental bending frequency is 5.27 Hz. This points to the fact that the vehicle crossing portion of the spectrum is helpful in identifying which modes are excited as the vehicle traverses the bridge.

Since the accelerometers are more sensitive than the deflection gages, the frequency resolution of an acceleration spectrum is usually greater than the resolution of the deflection spectrum. This may be seen by comparing Figure H.6a with Figure H.6b. If only the deflection spectrum

were used, the fundamental bending frequency would be taken as 3.32 Hz. However this 3.32 Hz spike in Figure H.6a actually contains both the fundamental bending and torsional modes which are shown separately in the acceleration spectrum in Figure H.6b. This phenomenon is also readily apparent in Figures H.8a and H.8b. Another factor contributing to greater frequency resolution is the number of data points decomposed. The more data points available for decomposition the greater the frequency resolution. This is important when the free vibration portion of the file decays very rapidly. Examples of this are the bridges in Category #7 which contain short span non-continuous reinforced concrete girder bridges. Figures H.16a through H.17c contain the free vibration spectra for these bridges and it is apparent that the resolution of the frequencies is not very great. One solution to this problem is decreasing the digitization sampling interval Δt, thus increasing the number of data points available for decomposition. However, this also increases storage space requirements both on magnetic tape and in the computer.

The spectra for two different bridges which had identical section properties and span lengths were included in the figures of Appendix H and Table 4.8 in order to ascertain if the modes of vibration were identical. The results may be seen by comparing the results for bridges CSB-B-1 and CSB-B-2 and also PCIB-A-1 and PCIB-A-2 in Table 4.8. For all practical purposes the excited frequencies or modes are identical for the two pairs of bridges.

It is beyond the scope of this investigation to identify all the frequencies which are excited by vehicles passing over the bridge. However, the fundamental bending and torsional frequencies are important and have been presented in Table 4.8 wherever possible. From a previous

analytical investigation(17) it was shown that the fundamental torsional frequency was close to and higher than the fundamental bending frequency. The special test "C" described in Appendix D served to verify this fact and helped to separate and identify these frequencies for three bridges, SB-C-1, KCSG-A-1 and CSB-C-1. Briefly, the test vehicle was driven at constant velocity down the centerline of each bridge in order to eliminate as much of the torsional effects as possible. At this same velocity the bus was then driven over the bridge in the normal travel lane and finally driven across the bridge as close to the curb as was safe. By comparing the frequency content from these three test runs, the torsional content for the bus in the curb lane should be significantly higher than that for the bus at the bridge centerline. The results of this special test are shown in Table 4.9 and Figures H.1a through H.3c. Table 4.9 presents the percentage mean square contribution of each frequency excited to the total mean square value for the free vibration file. The term f_b^1 designates the fundamental bending frequency while f_t^1 is the first torsional frequency. The results for both SB-C-1 and KCSG-A-1 show a marked increase in the contribution of the torsional mode to the total motion of the bridges while the contribution of all other frequencies either decreases or shows no trend. For the three span bridge CSB-C-1 the results are not as pronounced as for the two previous bridges but the trend of the contribution of the second frequency 4.49 Hz, shows an increase, while the contribution of the first frequency decreases between the results from the centerline (CL) and curb lane. There is no change in the contribution of the 5.86 Hz mode. Thus we have experimentally identified the fundamental torsional mode for each of the three aforementioned bridges. Upon the results of this test the fundamental torsional

Table 4.9. Identification of the Fundamental Torsional Frequency by Comparison of the Percentage of the Total Mean Square Contribution for Each Frequency Excited

Bridge	Frequency (Hz)	Transverse Vehicle Position		
		CL	Travel Lane	Curb Lane
SB-C-1	$f_b^1 = 4.88$	68	53	13
	$f_t^1 = 5.27$	14	18	57
	7.32	9	15	13
	12.01	7	13	15
		98%	99%	98%
KCSG-A-1	$f_b^1 = 2.34$	4	12	8
	$f_t^1 = 2.83$	3	7	54
	3.42	30	39	17
	3.80	2	6	6
	12.99	20	26	11
	13.77	33	1	1
		92%	91%	97%
CSB-C-1	$f_b^1 = 3.91$	85	86	77
	$f_t^1 = 4.49$	5	7	11
	5.86	1	1	1
		91%	94%	89%

frequency was identified for the remaining eighteen bridges and are shown in Table 4.8.

One of the significant findings of the frequency analysis is the number of modes excited by vehicles traversing a bridge together with the fact that the fundamental bending mode does not generally constitute the major portion of the overall bridge motion as was previously thought. The former finding is evident when looking at the last column of Table 4.8 or looking at the spectra in Figures H.4a through H.21c. A good example is the frequency spectrum shown in Figure H.7b. The percentage of the mean square value for each excited frequency to the total mean square value is as follows:

Frequency (Hz)	% of Total Mean Square
$f_b^1 = 2.15$	1.4
$f_t^1 = 2.73$	26.3
3.91	2.9
6.83	11.1
7.42	15.4
12.30	2.2
14.26	9.7
14.65	6.4
24.22	16.9
	92.3%

After the vehicle has left the bridge the fundamental bending mode is contributing approximately 1% of the bridge motion in terms of acceleration and 5% in terms of displacement (see Figure H.7a). If we consider a case where the vehicle is on the bridge, Figure H.14a indicates that the displacement is partially composed of the following:

Crawl Curve	27%
f_b^1 = 5.27 Hz	11%
9.77 Hz	30%
	68%

while the acceleration for the same vehicle crossing the bridge has the composition (see Figure H.14c)

f_b^1 = 5.27 Hz	4%
9.77 Hz	41%
14 Hz-17 Hz	25%
	70%

The preceding examples indicate that bridge motion is very complex and modes of vibration exist which are not explained by either common bending or torsional mode. In addition if we are considering human response to bridge motion in its relationship to frequency content, it can be seen that the contribution of some particular frequency to the total motion of the bridge is different depending upon whether the quantity under consideration is acceleration or displacement. Even though the displacement, velocity, acceleration and jerk files contain the same frequencies, the contribution of each mode is different for each quantity. Finally it should be noted that each frequency in a spectrum is not a combination of any other frequencies but is an independent quantity (see Appendix G, section G.2).

As was mentioned at the beginning of this section, the fundamental bending frequency corresponding to full composite action and to no composite action was calculated for the bridges which were not definitely composite construction and the results presented in Table 4.8. All of the single span bridges exhibit full composite action. The reason for the high fundamental bending frequency for SB-A-4 may be due to joint or bearing conditions. All of

the two span composite construction bridges also display full composite action with the calculated frequency comparing very closely with the measured values. The three span steel beam bridges CSB-A-2 through CSB-C-1 exhibit practically no composite action.

CHAPTER V: SUMMARY AND CONCLUSIONS

5.1. Summary of Test and Analysis Procedures

A sample of sixty-two bridges containing from one through four spans and of all types of construction was instrumented with a deflection gage and accelerometers and the response of these bridges under actual traffic conditions were recorded, digitized, and stored on magnetic tape. More than 2200 individual vehicle crossings were collected constituting over 13,000 deflection and acceleration records. Due to the cost of computer time used to reduce the data, only 900 vehicle crossings were actually digitized and stored on magnetic tape. Of these 900 vehicle crossings, approximately 65% were trucks, 30% were a test vehicle, and passenger cars and light delivery vans constituted the remaining 5%.

The main objective of this investigation was to establish the basic response characteristics for different types of highway bridge structures under the passage of ordinary vehicular traffic. The response characteristics under consideration were the maximum values of displacement, velocity, acceleration, and jerk together with the frequency of the excited modes of vibration as a vehicle would traverse a bridge. In order to achieve this goal, computer programs were developed which would furnish the desired response characteristics from the experimental data which was in the form of displacement and acceleration records. These computer programs together with their documentation are included in the Appendices of this paper. The following is a list of the functions performed by the programs:

(1) Calculate the log decrement and damping ratio of the bridge,

(2) Smooth the deflection file and differentiate it either once, twice or three times for the corresponding velocity, acceleration and jerk,

(3) Smooth the acceleration file and differentiate once for jerk,

(4) Integrate the acceleration file once or twice to obtain the corresponding velocity or displacement,

(5) Calculate the maximum values for displacement, velocity, acceleration and jerk, and

(6) Find the Fourier spectrum for either the vehicle crossing portion or free vibration portion of each data file.

Algorithms were developed to integrate the acceleration file to find either the velocity or displacement. Three different integration algorithms were presented, one for single span, one for two span, and one for three and four span bridges.

A differentiation technique was presented by which a raw data file was least squares fit to a particular smoothing polynomial and differentiated, thus eliminating the problem of compounding the errors introduced into the data during the recording and digitization process. For the sake of clarity, a 3rd degree smoothing polynomial will be referred to as $J=3$ and the number of data points used in the smoothing will be $(2xK)+1$. $K=3$ will denote seven consecutive data points being least squares fit to the specified polynomial with three data points either side of the point under consideration. For all types of bridges and vehicles the maximum acceleration or velocity may be obtained by averaging the maximum values obtained by smoothing the displacement with $J=3$, $K=4.5$ and differentiating twice. The maximum jerk may be obtained by averaging the

maximum values obtained by smoothing the corresponding acceleration with J=3, K=3, 6 and averaging the results after differentiation.

Using the techniques and computer programs just described, the maximum displacement, velocity, acceleration, jerk, and damping ratio were calculated for each digitized vehicle crossing. The results were presented in table form showing the mean, sample standard deviation and range for the maximum displacement, velocity, acceleration and jerk for each sub-category of bridges listed in Appendix A. The Fourier spectra of the displacement and acceleration file were presented for one bridge of each sub-category and the results were summarized in table form. The table included the following:

(1) theoretical fundamental bending frequency assuming
  (a) non-composite action where applicable
  (b) full composite action,
(2) measured fundamental bending frequency obtained from the Fourier spectra,
(3) percent composite action where applicable
(4) measured fundamental torsional frequency and
(5) additional dominant frequencies which were excited.

5.2. Summary of Results and Conclusions

One of the significant findings of this investigation was the level of acceleration experienced by the bridges. The highest acceleration experienced by any of the bridges was 133 in/sec^2, while the highest mean maximum acceleration was 74 in/sec^2 and there were only five instances where the maximum acceleration for a particular vehicle crossing was larger than 100 in/sec^2. If only heavy trucks are considered, the average of the mean maximum accelerations was 55 in/sec^2 for all types of steel bridges and 24 in/sec^2

for all types of concrete bridges. If we consider the acceleration criterion for human response to harmonic vertical vibration presented by Wright and Walker(26) the accelerations obtained from this study would fall between the "imperceptible" to "unpleasant to a few" categories.

The values for maximum velocity and jerk follow the same trend as maximum acceleration with respect to type of construction. The average of the mean maximum jerk for steel beam bridges was 3500 in/sec^3 and 1800 in/sec^3 for all types of concrete bridges. For velocity the values were 0.91 in/sec and 0.29 in/sec, respectively. For all types of construction the jerk increased significantly as the span length decreased while the velocity followed the same trend except for three-span continuous steel beam bridges where the mean maximum velocity increased as the span lengths increased. For this same type of bridge the maximum mean acceleration remained fairly constant as the span lengths increased instead of decreasing as was expected.

Using the test vehicle, the vehicle velocity versus the maximum displacement, velocity, acceleration and jerk was investigated for one through three span bridges. As the vehicle velocity increased so did the values for velocity, acceleration and jerk which was to be expected. For displacement the single and two span bridges indicated a decrease in maximum displacement with increasing vehicle velocity. The three span bridges indicate a slight increase in displacement with increasing vehicle velocity.

No single bridge exhibited significantly higher response values than other bridges within a bridge subcategory or category. If one type of bridge had to be singled out for high response characteristics it would be the two span (76', 76') composite continuous steel beam bridges KCSB-C. The maximum test program acceleration of 133 in/sec^2 occurred on this type of bridge with the mean

maximum acceleration being 74 in/sec^2. It is unfortunate that there were no velocity or jerk values available for this bridge type but based on past experience these values would also be relatively high.

The frequency analysis resulted in two important conclusions. The frequency spectra from all types of bridges indicated that bridge motion is composed of a number of modes of vibration some of which are not explained by either common bending or torsional mode shapes.

For instance, from two to five district dominate frequencies may appear between the fundamental torsional and second bending frequencies. The modes corresponding to these frequencies may constitute as much as 40% of the total bridge motion.

The second conclusion concerns the fundamental torsional mode of vibration. In general the first torsional mode contributes as much to the total motion of the bridge as does the fundamental bending mode and in quite a few cases exceeds the contribution of the fundamental bending by a significant amount. For example, there have been cases where the fundamental bending mode is contributing approximately 1% and the first torsional 25% to the total bridge motion.

The majority of fundamental bending frequencies for the bridges tested compared very well with the computed natural bending frequencies. All of the single span steel beam bridges exhibited full composite action while the 3-span continuous steel beam bridges displayed approximately 40% of full composite action.

Further study could be conducted to identify the various mode shapes which are not explained by the common bending or torsional shapes. Another area of study would be the use of the displacement data which has been collected during this test program to investigate the

maximum stresses occurring as a heavy vehicle crosses the bridge. The test data might also be useful in the area of fatigue study of bridge girders.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Shahabadi, Ali, "Bridge Vibration Studies", Ph.D. Thesis, Purdue University, W. Lafayette, Indiana, 1977.

2. Aramraks, T., "Highway Bridge Vibration Studies", Ph.D. Thesis, Purdue University, W. Lafayette, Indiana, 1975.

3. Lipkey, J. F. and Chuck, F. C., "Interim Report, Bridge Vibration Study", Indiana State Highway Commission, Research and Training Center, W. Lafayette, Indiana, 1975.

4. Varney, R. F. and Viner, J. G., "Vibration Studies Relating to the Failure of the Point Pleasant Bridge", A Journal of Highway Research, Public Roads, Vol. 36, No. 8, June, 1971.

5. Otnes, R. K. and Enochson, L., "Digital Time Series Analysis", John Wiley & Sons, Inc., New York, 1972.

6. Bendat, J. S. and Piersol, A. G., "Random Data: Analysis and Measurement Procedures", John Wiley & Sons, Inc., New York, 1971.

7. Bice, P. K., "Get Accurate Fast Fourier Transforms", Electronic Design, p. 87, May 10, 1973.

8. Wylie, C. R., "Advanced Engineering Mathematics", McGraw Hill Book Company, Inc., New York, 1960.

9. Casaletto, J. J., and Rice, J. R., "Smoothing and Estimating Derivatives of Equispaced Data", Purdue University Computer Sciences Department, CSD TR 35, May 1969.

10. "The Determination of True Ground Motion by Integration of Strong-Motion Records: A Symposium", Bulletin of the Seismological Society of America, Vol. 33, No. 1, January 1943.

11. Housner, G. W., "Ground Displacement Computed from Strong-Motion Accelerograms", Bulletin of the Seismological Society of America, Vol. 37, No. 4, October 1947.

12. Berg, G. V., and Housner, G. W., "Integrated Velocity and Displacement of Strong Earthquake Ground Motion", Bulletin of the Seismological Society of America, Vol. 51, No. 2, April 1961.

13. Nigam, N. C. and Jennings, P. C., "Digital Calculation of Response Spectra from Strong-Motion Earthquake Records", California Institute of Technology, Earthquake Engineering Research Laboratory, June 1968.

14. Ruiz, P., and Penzien, J., "Probabilistic Study of the Behavior of Structures During Earthquakes", Earthquake Engineering Research Center, University of California, Report No. EERC 69-3, March 1969.

15. "Strong Motion Earthquake Accelerograms, Digitized and Plotted Data", California Institute of Technology, Earthquake Engineering Research Laboratory, Report No. EERL 71-50, September 1971.

16. Fenves, S. J., Veletsos, A. S. and Siess, C. P., "Dynamic Studies of Bridges on the AASHO Road Test", Highway Research Board Special Report 71, National Academy of Sciences, Washington, D.C., 1962.

17. Eberhardt, A. C., "A Finite Element Approach to the Dynamic Analysis of Continuous Highway Bridges", Ph.D. Thesis, Univ. of Illinois, Urbana, Illinois, 1972.

18. Fenves, S. J., Veletsos, A. S. and Siess, C. P., "Dynamic Studies of the AASHO Road Test Bridges", Highway Research Board Special Report 73, National Academy of Sciences, Washington, D.C., 1962.

19. Huang, T., "Dynamic Response of Three Span Continuous Highway Bridges", Ph.D. Thesis, Univ. of Illinois, Urbana, Illinois, 1960.

20. Oran, C. and Veletsos, A. S., "Analysis of Static and Dynamic Response of Simple-Span, Multigirder Highway Bridges", Civil Engineering Studies, Structural Research Sieries No. 221, Univ. of Illinois, Urbana, Illinois, July 1961.

21. Nieto-Ramirez, J. A. and Veletsos, A. S., "Response of Three-Span Continuous Highway Bridges to Moving Vehicles", Civil Engineering Studies, Structural Research Series No. 276, Univ. of Illinois, Urbana, Illinois, January 1964.

22. Csagoly, P. F., Campbell, T. I. and Agarwell, A. C., "Bridge Vibration Study", The Engineering Research Branch of the Ministry of Transportation and Commerce, Ontario, September 1972.

23. Lee, J. A. N., Brown, R., Windover, L., "Highway Bridge Vibrations III, Cantilever-Type Structures", Ontario JHRP, Report No. 39, Ontario, Canada, January 1966.

24. Wright, D. T. and Green, R., "Highway Bridge Vibrations, Part II", Ontario JHRP, Report No. 5, Ontario, Canada, May 1964.

25. Linger, D. A. and Hulsbos, C. L., "Dynamic Load Distribution in Continuous I-Beam Highway Bridges", Highway Research Record, No. 34, 1963.

26. Wright, R. N. and Walker, W. H., "Vibration and Deflection of Steel Bridges", Engineering Journal, American Institute of Steel Construction, Vol. 9, No. 1, January 1972.

27. Mason, A. F. and Duncan, M. A. G., "Composite Steel and Concrete Bridge Construction in Southern Rhodesia", Institution of Civil Engineers, Proceedings, London, April 1962.

28. Wen, R. K. and Veletsos, A. S., "Dynamic Behavior of Simple-Span Highway Bridges", Highway Research Board, Bulletin 315, 1962.

29. Linger, D. A. and Hulsbos, C. L., "Dynamic Tests of a Three-Span Continuous I-Beam Highway Bridge", Highway Research Board, Bulletin 339, 1962.

30. Nebraska Department of Roads Bridge Design Section, "Dynamic Test of Two Cantilever Type Deck Steel Girder Bridges", August 1961.

31. Fleming, J. F. and Romualdi, J. T., "Dynamic Response of Highway Bridges", Journal of the Structural Division, ASCE, Vol. 87, October 1961.

32. Hulsbos, C. L. and Linger, D. A., "Dynamic Tests of a Three-Span Continuous I-Beam Highway Bridge", Highway Research Board, Bulletin 279, 1960.

33. Oehler, L. T., "Vibration Susceptibilities of Various Highway Bridge Types", Journal of the Structural Division, ASCE, Paper 1318, July 1957.

34. Rowe, R. W., "Vibration in Bridges", Concrete Bridge Design, Chapter 13, London, 1962.

35. Standard Specification for Highway Bridges, American Association of State Highway Officials, 10th Edition, Washington, D.C., 1969.

36. Veletsos, A. S. and Huang, T., "Analysis of Dynamic Response of Highway Bridges", Journal of the Engineering Mechanics Division, ASCE, Vol. 96, No. EM5, October 1970.

37. "The AASHO Road Test, Report 4 - Bridge Research", Highway Research Board Special Report, No. 61D, 1962.

38. "Tractor-Trailer Ride", Technical Board of Society of Automotive Engineers, Inc., 485 Lexington Ave., New York, N.Y., May 1956.

NOTICE

The following Appendices listed in the Table of Contents of this Report have not been included in this copy of the Report.

Appendix	Title	Pages
A	List of Bridge Structures Tested	100-106
B	Test Vehicle Parameters	107-110
C	Test Apparatus	111-118
D	Special Test Procedures	119-121
E	Digitization of the Test Data	122-152
F	Computer Program and Techniques Used to Analyze the Data	153-204
G	Frequency Response Analysis	205-229
H	Fourier Spectra of the Test Data	230-288

A recipient of the Report may secure a copy of the Appendix desired upon request to:

Joint Highway Research Project
Civil Engineering Building
Purdue University
West Lafayette, Indiana 47907

COVER DESIGN BY ALDO GIORGINI